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# **MGS Flood - Proprietary Version Users Manual**

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## **A Continuous Hydrological Simulation Model for Stormwater Facility Analysis for Western Washington**



**MGS**

Software LLC

**7326 Boston Harbor Road NE  
Olympia, WA 98506**

**Version 3.x  
June 30, 2005**

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*By*



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# **PART I – PROGRAM BACKGROUND INFORMATION**

## **1 INTRODUCTION**

MGSFlood is a general, continuous, rainfall-runoff computer model developed for the Washington State Department of Transportation specifically for stormwater facility design in Western Washington. The program uses the Hydrological Simulation Program-Fortran (HSPF)<sup>26</sup> routine for computing runoff from rainfall. The public domain version of the program includes a routing routine that uses a stage-storage-discharge rating table to define a stormwater retention/detention facility or reservoir, routines for computing streamflow magnitude-frequency and duration statistics, and graphics routines for plotting hydrographs and streamflow frequency and duration characteristics. The program meets the requirements of the 2005 Washington State Department of Ecology Stormwater Management Manual for Western Washington<sup>9</sup>.

This manual documents the proprietary version of MGSFlood, which includes several features and capabilities that were added to the public-domain version of MGSFlood<sup>27</sup>. The proprietary version includes routines for:

- The ability to simulate runoff and route flows through a network consisting of multiple structures. The structures include; detention/retention ponds, sand filters, infiltration trenches, open channels, flow splitters, and user specified rating tables
- The ability to simulate lateral flow connections and the infiltration of runoff through dispersion
- Infiltration that accounts for the proximity of the groundwater table, as defined by Massmann<sup>30</sup>
- The simulation of soils that become saturated due to high groundwater
- The ability to perform wetland hydroperiod analyses according to criteria developed by the Puget Sound Wetland and Stormwater Management Program
- Automatically sizing stormwater ponds, infiltration ponds, and infiltration trenches to meet the Ecology flow-duration standards in a manner that seeks to minimize the size of the structure.

## **2 MGS Flood Model Applicability, Limitations and Program Configuration**

### **2.1 Model Applicability and Limitations**

MGSFlood is intended for the analysis of stormwater detention facilities in the lowlands of western Washington. The program utilizes the HSPF routines for computing runoff from rainfall for pervious and impervious land areas. The program does not include routines for simulating the accumulation and melt of snow and its use should be limited to lowland areas

where snowmelt is typically not a major contributor to floods or to the annual runoff volume. In general, these conditions correspond to an elevation below approximately 1500 feet.

The program was designed for the analysis of stormwater for small-scale development and is applicable to either road or land development projects. The program is currently provided with precipitation timeseries at a 1-hour timestep, which is appropriate for stormwater detention design only. Precipitation input at a shorter timestep (15-minutes) would be needed to accurately estimate the peak runoff rate needed for sizing of conveyance structures for small-scale development projects and the design of water quality treatment facilities that depend on runoff rate. Thus, the program should not be used for conveyance design unless the conveyance system is downstream of a stormwater detention pond, where regulation of streamflow renders a 1-hour time-step sufficiently accurate for conveyance design.

The program provides acceptable results for sites up to about 320 acres in size (one-half square mile) without the inclusion of channel routing. The attenuating effect of channel routing on flood peak discharge becomes increasingly more important as the size of the watershed and complexity of the stream network increases. For sites larger than 320 acres, then channel routing should be included between runoff nodes.

## **2.2 Disclaimer**

MGSFlood is a complex program that requires engineering expertise to use correctly. MGS Software LLC assumes absolutely no responsibility for the correct use of this program. All results obtained should be carefully examined by an experienced professional engineer to determine if they are reasonable and accurate.

Although MGS Software LLC has endeavored to make this program error free, the program is not and cannot be certified as infallible. Therefore, MGS Software LLC makes no warranty, either implicit or explicit, as to the correct performance or accuracy of this software.

In no event shall MGS Software LLC be liable to anyone for special, collateral, incidental, or consequential damages in connection with or arising out of use of this program.

## **2.3 Program Configuration**

Figure 2.1 shows a schematic of the MGSFlood modeling package. The main program module, MGSFlood.exe, controls the user interface, HSPF, statistics, routing, and pond optimization routines. When the program starts, the location of the program and project subdirectories on the computer system is read from the Windows Registry. Project data files have a user specified name and a *.fld* extension. These files are Microsoft Access database files and are stored in the project subdirectory.

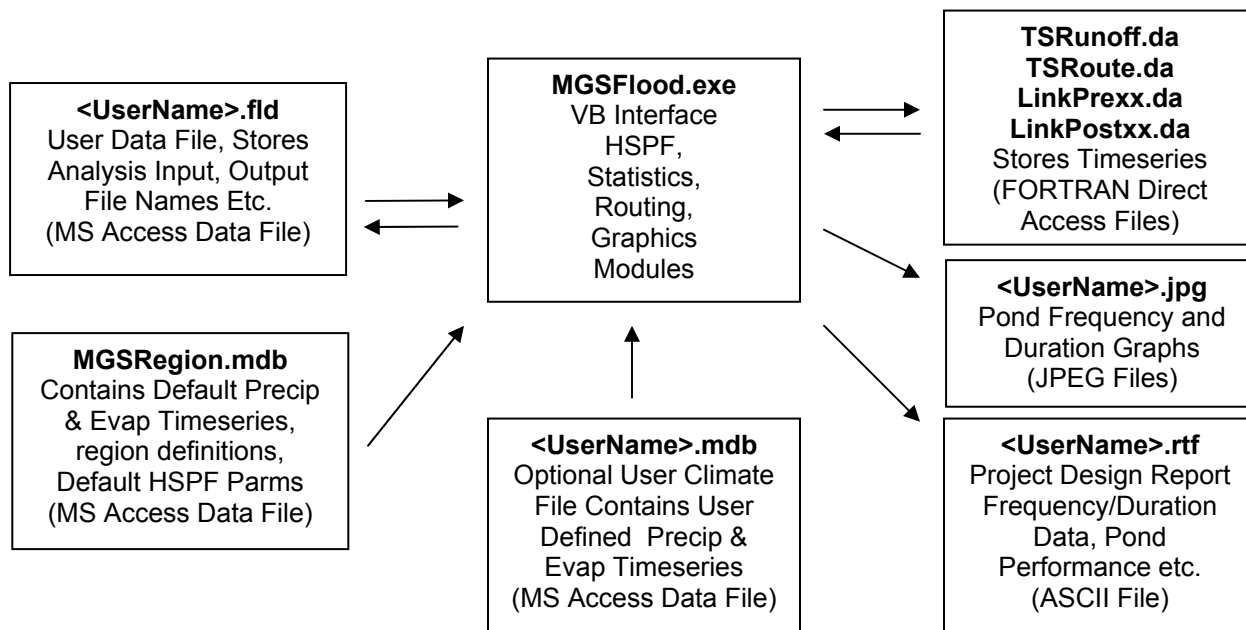


Figure 2.1 – MGSFlood Model Components

## 2.4 Precipitation and Evaporation Input

MGSRegions.mdb is an Access database file that contains the precipitation and evaporation timeseries for each region, and the default HSPF parameters.

## 2.5 Runoff Data File

Runoff is computed by MGSFlood using the HSPF<sup>26</sup> library routine. Precipitation and evaporation are read from the MGSRegion.mdb file, runoff is computed for predevelopment and postdevelopment conditions, and saved to FORTRAN, binary, direct access files called **TSRunoff.da**, **TSRoute.da**, **LinkxxPre**, and **LinkxxPost**. The same FORTRAN direct access files are overwritten for each project analyzed by the flood model, i.e. the computed runoff timeseries are not saved for each project. Thus, the project runoff must be recomputed to ensure that the files are up-to-date and contains runoff for the project currently under consideration.

## 2.6 Project Documentation and Graphics Files

Project documentation is stored in a Windows Rich Text File format in the project subdirectory with an *.rtf* extension. These files may be read using Microsoft Word or WordPad. This file is created/overwritten each time the report is written by the program.

Files containing images of graphs plotted on the screen are also stored in the project subdirectory. These files are *JPEG* format and contain the latest images of hydrograph, flood frequency, and flow duration plots generated by the program. The JPEG graphic images can be imported into any software that accepts the JPEG format. This feature is intended to support importing graphics into word processing programs for preparation of reports and other documents.



### 3 HSPF Runoff Routine and Runoff Parameters

MGS Flood uses the rainfall-runoff routines from Version 12 of the Hydrological Simulation Program-Fortran (HSPF)<sup>26</sup>. HSPF uses multi-year inputs of hourly precipitation and evaporation, keeps a running accounting of the moisture within the soil column and in groundwater storage, and simulates a multi-year timeseries of hourly runoff.

#### 3.1 Pervious Land Parameters

Default HSPF model parameters that define interception, infiltration, and movement of moisture through the soil, are based on work by the USGS<sup>7,8</sup> and King County<sup>17</sup>. Pervious areas have been grouped into three land cover categories; forest, pasture, and lawn, and three soil/geologic categories; till, outwash, and saturated/wetland soil for a total of seven cover/soil type combinations as shown in Table 3.1. The combinations of soil type and land cover are called *pervious land segments* or *PERLNDs*. Default runoff parameters for each PERLND are summarized in Table 3.2. These values are loaded automatically by the program for each project. If these values are changed by the user, the changed values are noted in the project documentation report (See Section 15).

**Table 3.1 - Pervious Land Soil Type/Cover Combinations used with HSPF Model Parameters**

Pervious Land Soil Type/Cover Combinations	
1.	Till/Forest
2.	Till/Pasture
3.	Till/Lawn
4.	Outwash/Forest
5.	Outwash/Pasture
6.	Outwash/Lawn
7.	Saturated Soil/All Cover Groups

**Table 3.2 – Default Runoff Parameters for Each Pervious Land Segment (PERLND)**

Parameter	Pervious Land Segment (PERLND)						
	Till Soil			Outwash Soil			Saturated Soil
	Forest	Pasture	Lawn	Forest	Pasture	Lawn	Forest/Pasture/ or Lawn
LZSN	4.5	4.5	4.5	5.0	5.0	5.0	4.0
INFILT	0.08	0.06	0.03	2.0	1.6	0.8	2.0
LSUR	400	400	400	400	400	400	100
SLSUR	0.1	0.1	0.1	0.05	0.05	0.05	0.001
KVARY	0.5	0.5	0.5	0.3	0.3	0.3	0.5
AGWRC	0.996	0.996	0.996	0.996	0.996	0.996	0.996
INFEXP	2.0	2.0	2.0	2.0	2.0	2.0	10.0
INFILD	2.0	2.0	2.0	2.0	2.0	2.0	2.0
BASETP	0.0	0.0	0.0	0.0	0.0	0.0	0.0
AGWETP	0.0	0.0	0.0	0.0	0.0	0.0	0.7
CEPSC	0.2	0.15	0.1	0.2	0.15	0.1	0.1
UZSN	0.5	0.4	0.25	0.5	0.5	0.5	3.0
NSUR	0.35	0.3	0.25	0.35	0.3	0.25	0.5
INTFW	6.0	6.0	6.0	0.0	0.0	0.0	1.0
IRC	0.5	0.5	0.5	0.7	0.7	0.7	0.7
LZETP	0.7	0.4	0.25	0.7	0.4	0.25	0.8

PERLND parameter definitions:

LZSN =lower zone storage nominal (inches)

INFILT =infiltration capacity (inches/hour)

LSUR =length of surface overland flow plane (feet)

SLSUR =slope of surface overland flow plane (feet/feet)

KVARY =groundwater exponent variable (inch<sup>-1</sup>)

AGWRC =active groundwater recession constant (day<sup>-1</sup>)

INFEXP =infiltration exponent

INFILD =ratio of maximum to mean infiltration

BASETP =base flow evapotranspiration (fraction)

AGWETP =active groundwater evapotranspiration (fraction)

CEPSC =interception storage (inches)

UZSN =upper zone storage nominal (inches)

NSUR =roughness of surface overland flow plane (Manning 's n)

INTFW =interflow index

IRC =interflow recession constant (day<sup>-1</sup>)

LZETP =lower zone evapotranspiration (fraction)

A complete description of the PERLND parameters can be found in the HSPF User Manual<sup>26</sup>. Default PERLND Parameters used in the model were developed for the Puget Sound Lowlands by the US Geological Survey<sup>7,8</sup>

### 3.2 User Defined Pervious Land Segments

Two additional Pervious Land Segments (PERLNDs) may be specified by the user by opening the HSPF Parameter sheet and clicking the *User* button at the bottom of the page. A window will appear with parameter fields for up to two additional PERLNDs. The user can specify the name of these as well as the HSPF parameters. This feature allows the user to define land cover/soil type combinations not included in the default parameters. Once the “Active” checkbox is checked and the parameters have been defined for each user-defined PERLND, the user-defined PERLND will be active on the subbasin area window.

### 3.3 High Groundwater Simulation

The model includes the high groundwater simulation option present in HSPF Version 12<sup>26</sup>. This routine was designed for use with soils that can be influenced by seasonal high groundwater that approaches or rises above the ground surface. This method, which is applicable for wetlands and low gradient areas, is a modified version of the standard hydrologic method in HSPF (i.e., the Stanford Watershed Model). The routine converts the conceptual storages in the HSPF PERLND routine to groundwater elevation above an arbitrary datum using soil properties such as porosity. A summary of the routine is described below and the reader is referred to the HSPF Version 12 Users Manual<sup>26</sup> for a more detailed discussion.

To represent the high water table/low gradient environment, it is necessary to keep track of the groundwater levels (saturated zone elevation) and to model the interaction between the saturated zone and the unsaturated zone. The interaction between the saturated and unsaturated zones corresponds to transfers between the groundwater storage and the other storages. Determining when and how the rising groundwater starts affecting the other storages requires being more specific about the location and capacity of the storages. The porosity of a soil is the volume of pore space as a fraction, or a percentage, of the total soil volume. Porosity varies, with typical values for sand of about 40%, and higher values for silts and clays.

Water is stored in soil as adhesion water, cohesion water, and gravitational water. Adhesion water is electrically bonded to soil particles and is immobile except at very high temperatures (in drying ovens). Cohesion water is bonded in soil by capillary forces and weaker electrical forces. Cohesion water is roughly equal to the “available water”, the difference between the wilting point and field capacity. Gravitational water will drain from soils in the unsaturated zone unless drainage is inhibited. Gravitational water can be defined to be present in macropores (cohesion water is present in micropores).

For modeling purposes, the total porosity is divided into porosity in micropores (PCW, cohesion water), and porosity in macropores (PGW, gravitational water). The upper layer of the soil may be disturbed and have a larger porosity in macropores (UPGW). The porosity of micropores is assumed to be the same throughout the soil column. Cohesion water is stored in the lower zone storage, while gravitational water is stored in the upper zone and interflow storages.

$P_{cw}$  is the porosity of cohesion water and  $P_{gw}$  is the porosity of gravity water (Figure 3.1).

The “groundwater level” is the elevation of the saturated zone above an arbitrary datum such as mean sea level. The active groundwater storage is gravity water stored above the minimum channel or canal elevation that is within or adjacent to the land.

When the groundwater level reaches the surface, any additional water is assigned to surface detention storage and the surface detention storage represents the depth of inundation over the land segment. Evaporation from the surface storage takes place at the potential rate after interception storage has been exhausted and runoff is defined based on the surface hydraulics of the wetland. The program defines wetland surface water storage with a user defined elevation-volume table and outlet hydraulics using the structures defined in Section 8.

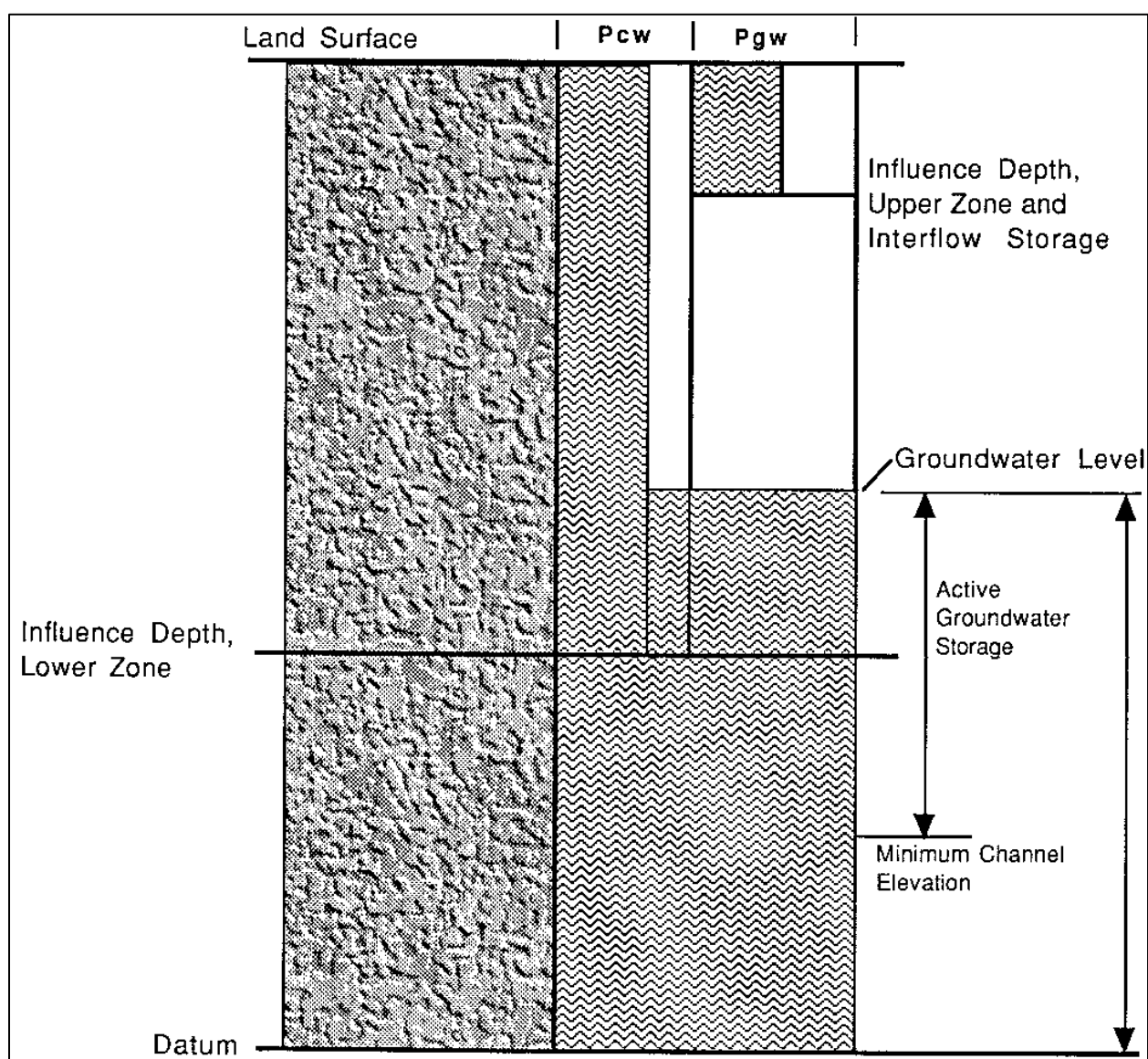


Figure 3.1 – Sketch of Soil Moisture in the Unsaturated Zone Under High Water Table Conditions

### 3.4 Impervious Land Parameters

Default runoff parameters for impervious surface, called *IMPLNDs* are summarized in Table 3.3.

**Table 3.3 – Impervious Cover (IMPLND) Parameters**

Parameter	Value
LSUR	100
SLSUR	0.01
NSUR	0.1
RETSC	0.1

IMPLND Parameter Definitions:

LSUR = length of surface overland flow plane (feet)

SLSUR = slope of surface overland flow plane (feet/feet)

NSUR = roughness of surface overland flow plane (Manning 's n)

RETSC = retention storage (inches)

A complete description of the IMPLND parameters can be found in the HSPF User Manual<sup>26</sup>. IMPLND Parameters were developed for the Puget Sound Lowlands by the US Geological Survey<sup>7,8</sup>.

### 3.5 Determining PERLND Soil Type from SCS Soil Mapping

The soils at the project site must be classified into one of the three default categories for use in the MGSFlood model. These soils categories are: till, outwash, or saturated soil, as defined by the USGS<sup>7,8</sup>.

Soils formed in areas with glacial till are underlain at shallow depths by relatively impermeable glacial till (also known as “hard-pan”). Glacial till deposits contain large percentages of silt or clay and have low percolation rates. Only a small fraction of infiltrated precipitation reaches the groundwater table through the till. The rest moves laterally through the thin surface soil above the till deposit as interflow. Shallow soils over bedrock should also be classified as till soils because the hydrologic response from these areas is similar to till.

Soils formed in areas with glacial outwash deposits consist of sand and gravels that have high infiltration rates. The majority of rainfall is infiltrated and percolates to the groundwater table in these areas. Creeks draining outwash deposits often intersect the groundwater table and receive most of their flow from groundwater discharge. Site developments in outwash areas are typically located higher in the watershed and groundwater discharge is not present. Thus, groundwater is typically not included in runoff calculations in outwash (or till) areas.

Wetland soils remain saturated throughout much of the year. The hydrologic response from wetlands is variable depending on the underlying geology, the proximity of the wetland to the regional groundwater table, and the bathymetry of the wetland. Generally, wetlands provide some baseflow to streams in the summer months and attenuate storm flows via temporary storage and slow release in the winter.

Mapping of soil types by the Soil Conservation Service (SCS, now the National Resource Conservation Service (NRCS)) is the most common source of soil/geologic information used in hydrologic analyses for stormwater facility design. Each soil type defined by the SCS has been classified into one of four hydrologic soil groups; A, B, C, and D. As is common practice in hydrologic modeling in western Washington, the soil groups used in the MGSFlood model generally correspond to the SCS hydrologic soil groups as shown in Table 3.4.

**Table 3.4 – Relationship Between SCS Hydrologic Soil Group and MGS Flood Soil Group**

SCS	MGS Group
A	Outwash
B	Till or Outwash
C	Till
D	Wetland

SCS Type B soils can be classified as either glacial till or outwash depending on the type of soil under consideration. Type B soils underlain by glacial till or bedrock, or have a seasonally high water table would be classified as till. Conversely, well-drained B type soils would be classified as outwash.

The Ecology Stormwater Management Manual for Western Washington<sup>9</sup> relates SCS hydrologic soil groups to HSPF soil/geologic groups as shown in Table 3.5

**Table 3.5 – Relationship between SCS and HSPF Soil Groups**

SCS Hydrologic Soil Group	MGSFlood/HSPF Soil/Geologic Group
A/B	Outwash
C	Till
D	Wetland

## **4 Precipitation Input**

MGSFlood uses multi-year inputs of hourly precipitation and evaporation to compute a multi-year timeseries of runoff from the site. Using precipitation input that is representative of the site under consideration is critical for the accurate computation of runoff and the design of stormwater facilities. Precipitation and evaporation timeseries have been assembled for most areas of western Washington and are stored in a database file accessed by the program. These timeseries should be used for stormwater facility design.

### **4.1 Selection and Scaling of Precipitation for Stormwater Facility Design**

Accurate assessment of streamflow characteristics at a particular site is dependent upon numerous watershed and hydrometeorological factors. Among those factors, it is critically important to have a precipitation timeseries representative of the climatic and storm characteristics at the site of interest. However, it is rare that a long precipitation timeseries is available at the site of interest. This problem is commonly addressed by transposing the timeseries record from a “nearby” gage to the site of interest using some type of scaling routine to account for the differences in storm characteristics at the source and target sites.

Proper transposition is a very complex problem as storm characteristics vary by both duration and physical topographic setting across western Washington. For example, a site with a mean annual precipitation of 50-inches to the west of central Puget Sound has different precipitation magnitude-frequency characteristics than a site with 50-inches mean annual precipitation located to the east of central Puget Sound. Ideally, a dense network of hourly precipitation gages would be available and only minor amounts of scaling would be needed. Unfortunately, only a limited number of long-term, high quality hourly precipitation recording stations are available in western Washington. Therefore, the transposition of timeseries by scaling is a critical aspect of obtaining a representative timeseries for most sites.

Two methods of transposing precipitation timeseries are available in the MGSFlood model. The first method utilizes a family of pre-scaled precipitation and evaporation timeseries. These precipitation timeseries were developed using statistical scaling functions to scale hourly precipitation amounts for eight selected inter-durations within the timeseries. This method was used to produce “extended precipitation timeseries” with record lengths in excess of 100-years by combining and scaling precipitation records from widely separated stations<sup>21,22</sup>. Extended hourly precipitation and evaporation timeseries have been developed using this method for most of the lowland areas of western Washington where stormwater projects will be constructed. These timeseries should be used for facility design for projects located in the region shown in Figure 4.1.

For projects sites located outside of the extended timeseries region, a second precipitation scaling method is used. This method uses a simple scaling procedure that scales all hourly precipitation amounts in the source timeseries by a common scaling factor. Use of these two methods is described in the following sections.



timeseries can be found in the report; *Extended Precipitation Time-Series for Continuous Hydrological Model in Western Washington*, MGS Engineering Consultants, Inc., 2002<sup>20</sup>.

### **Recommended Applicability of Extended Precipitation Time-Series**

Extended precipitation time-series are preferred to precipitation time-series obtained from simple scaling procedures for all locations in the lowlands of western Washington where extended time-series are available. Extended time-series are preferred for a number of reasons as discussed in the following sections.

### **Multiple Scaling Functions**

Extended precipitation time-series are developed using a series of statistical scaling functions rather than a single scaling factor. These scaling functions provide for scaling of precipitation maxima at the 2-hour, 6-hour, 24-hour, 72-hour, 10-day, 30-day, 90-day and annual durations. This scaling is done in a manner to match the storm statistics (magnitude-frequency characteristics) expected for a given climatological setting based on regional analyses of the time-series records at over 50 precipitation gages located in the lowlands of western Washington. Thus, the storm characteristics are based on a very large sample-set of storms and stations rather than the record from a single station.

### **Scaling Difficulties Due to Complex Nature of Storm Characteristics**

Storm characteristics vary by duration, storm type, and season in western Washington. This complex behavior includes: short-duration, high-intensity storm events in the warm season; intermediate-duration, moderate intensity events in the early-fall through early-winter season; and long-duration, low intensity storm events in the late-fall through winter season. Multi-day through weekly periods of heavy precipitation are important events for rainfall-runoff modeling of forested conditions where the runoff response is primarily interflow and subsurface flow. Monthly and multi-month precipitation can also be important because they affect soil moisture conditions antecedent to storm events. Each of these durations and associated storm types has distinctive magnitude-frequency relationships that must be preserved in the scaling operation. Therefore, proper scaling of precipitation time-series must be accomplished at a wide range of durations to preserve the storm characteristics that are important for continuous rainfall-runoff modeling. Preservation of storm characteristics at numerous durations is not possible with a single scaling factor.

### **Storm Characteristics Vary by Topographic/Climatological Setting**

Storm characteristics also vary by topographic and climatological setting in western Washington. For example, storm characteristics and statistics are different for sites to the west of central Puget Sound in areas where mean annual precipitation is decreasing from west to east relative to sites east of central Puget Sound where mean annual precipitation is increasing from west to east. These two regions may be considered as being leeward of the Olympic Mountains and windward of the Cascade Mountains, respectively. This situation was specifically addressed in development of the extended precipitation time-series by providing separate time-series for west and east of central Puget Sound. This complex situation is more difficult to address when using a single scaling factor approach for stations that are randomly spaced throughout western Washington.

### **Diversity of Storm Temporal Patterns**

The long 121-year and 158-year records provide a rich diversity of the storm temporal patterns, multi-day sequences of storms, and seasonality of occurrence of storm events that are possible in western Washington. These long records represent three to five times the number of combinations of storm magnitudes and storm patterns that are typically available in the record from a single station. Long records with a diversity of storm temporal patterns provide for a robust examination of the performance of detention and water-quality facilities over a very wide range of flow conditions.

### **Estimation of Moderate to Rare Floods**

Estimation of rare flood events is always of interest in hydrologic modeling. Use of the extended record allows for interpolation rather than extrapolation in estimating the characteristics of 25-year, 50-year, and 100-year floods. This is particularly important for estimation of the flood magnitude-frequency characteristics of streamflows downstream of detention facilities, as these streamflows are not amenable to standard statistical frequency analysis.

### **Quality and Resolution of Precipitation Records**

The quality of precipitation records at hourly recording gages varies widely. The operators for most hourly gages are volunteers, who are part of the national cooperative network for precipitation measurement. High quality precipitation records are dependent upon both proper mechanical operation and diligent monitoring by the operator. Quality can be compromised by mechanical problems, poor maintenance, inattention to malfunctions, misoperation, and poor record keeping. Unfortunately, it is all too common to have numerous episodes of both short and prolonged periods of missing data in the records from many of the hourly gages in western Washington.

Hourly precipitation records from 1948 to the late 1960's were recorded by weighing bucket gages with paper strip charts. These gages provided for a resolution to 0.01-inches. In the late 1960's, these gages were replaced by tipping bucket gages. The vast majority of the replacement gages have a bucket volume that provides resolution to 0.10-inches, which gives poor temporal resolution for the low to moderate-intensity winter storms common in western Washington. Tipping bucket gages with 0.10-inch buckets are also susceptible to evaporation losses from partially full buckets between storm events. This can lead to significant underestimation of monthly and annual precipitation totals.

The extended precipitation-time-series records were developed from gages with very high quality records, operated by weather service personnel, and that measured to a resolution of 0.01-inches or higher.

### Extended Precipitation Timeseries Selection Example

A project site is located in Thurston County as shown in Figure 4.2. The Project Site is located in the Puget Sound West region with a mean annual precipitation of 51 inches. From the Climatic Region drop down box, select the extended precipitation timeseries for the western Puget Sound Region with mean annual precipitation closest to the project site. In this case, select Puget Sound West Region, 52 inches MAP. The mean annual precipitation may also be determined by entering the project latitude and longitude in the Mean Annual Precipitation Calculator (in decimal degrees) and clicking the *Compute MAP* button.



Figure 4.2 – Extended Precipitation Timeseries Selection Example

#### 4.1.2 Single Scaling Factor Approach

For projects sites located outside of the extended timeseries region, a *source* gage is selected and a single scaling factor is applied to transpose the hourly record to the site of interest (target site). The current approach for single scaling, as recommended in the *Stormwater Management Manual for Western Washington*<sup>9</sup>, is to compute the scaling factor as the ratio of the 25-year 24-hour precipitation<sup>21</sup> for the target and source sites:

$$\text{Scale Factor} = P_{25 \text{ TargetSite}} / P_{25 \text{ SourceGage}} \quad (4.1)$$

where:  $P_{25 \text{ TargetSite}}$  = 25-year 24-hour precipitation at the project site of interest (entered by user)

$P_{25 \text{ SourceGage}}$  = 25-year 24-hour precipitation at the source gage (provided by program)

The values of the 25-year 24-hour precipitation may be obtained from NOAA Atlas #2<sup>18</sup> or from the recently released WSDOT update of precipitation-frequency information for western Washington<sup>21</sup>. To utilize the recently updated precipitation-frequency information for western Washington, open the *Precipitation Map* from the *Project Location Tab*. Regions of influence for each gage are identified on the map along with the 25-year 24-hour

precipitation. Choose the precipitation region where the project site is located. Read the project site 25-year 24-hour precipitation from the map and enter it in the appropriate field on the Project Location Tab. The computed scale factor will be displayed in the Scale Factor field. Alternatively, the project 25-year 24-hour precipitation may be computed by entering the project latitude and longitude in the Precip Calculator (in decimal degrees) and clicking the *Compute 25-Yr. 24-Hr* button.

### Precipitation Input Selection Example

A project site is located in Grays Harbor County as shown in Figure 4.3. The project is located outside of the region where extended precipitation timeseries are available and the simple scaling approach must be used. The project site is located in the Clearwater precipitation region. The 25-year 24-hour precipitation at the Project Site is 6.0 inches. The Clearwater gage should be selected as the source for this project, and a project site 25-year, 24-hour precipitation of 6.0 inches should be entered in the appropriate field on the Project Location tab. The Scale factor would be computed by the program as the ratio of the project site to station 25-year, 24-hour precipitation, or 6.0 inches divided by 7.9 inches equals 0.759. This value would be displayed in the *Scale Factor* field and all precipitation values subsequently read by the program would be multiplied by this value.

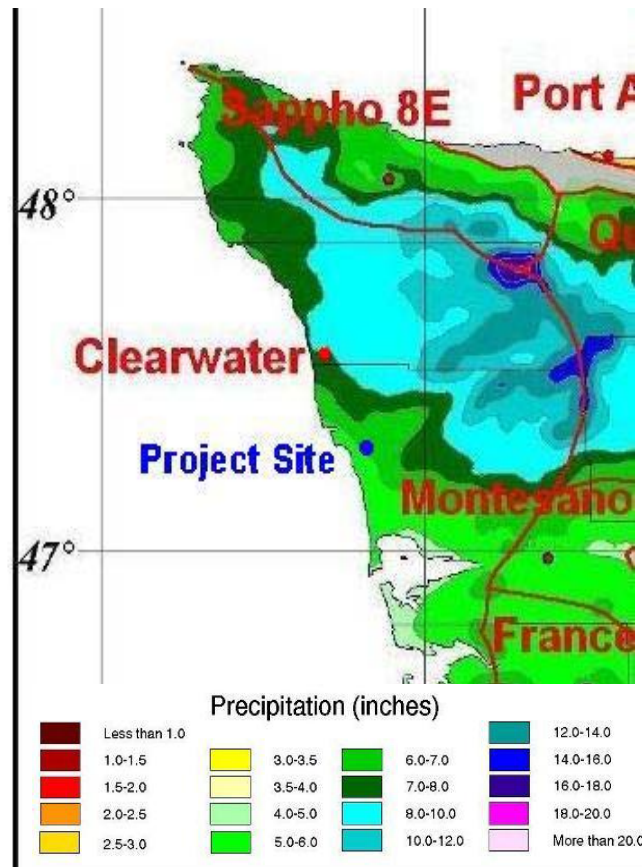


Figure 4.3 – Precipitation Input Selection Example for Project Sites Located Outside of Extended Precipitation Timeseries Region

## 5 Subbasin, Land Use, and Node Connection Input

To facilitate rainfall-runoff modeling, the project watershed must be defined in terms of subbasins, and the stream network within the watershed is described by a series of nodes connected by links (Figure 5.1). Land cover and soil type can vary within a subbasin and the program conducts rainfall-runoff modeling for each land cover/soil type combination separately. Nodes are used to collect runoff from the tributary area for a given subbasin and from the nodes of upstream subbasins. Links are used to connect one node to another and may be defined as channels, ponds, wetlands, infiltration trenches, etc. Link definitions are discussed in Section 6, Network Connections.

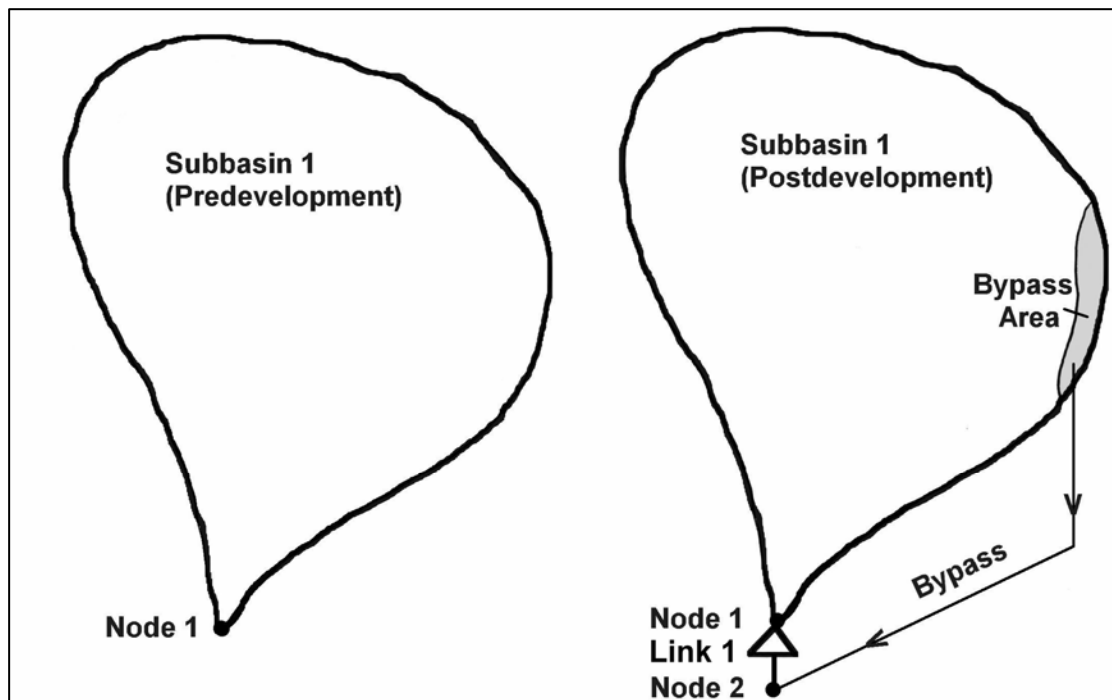


Figure 5.1 – Subbasin and Node Delineation, Single Subbasin with Bypass

### 5.1 Subbasin Land Use Input

Currently, the project tributary area can be divided into a maximum of six subbasins, however, modifications to the code are currently underway that would allow any number of subbasins, links, and nodes. The total acreage of each predeveloped and postdeveloped land cover/soil type combination (PERLNDs) is entered for each subbasin. A tributary node number is then chosen and all of the computed runoff from the subbasin is collected at this node.

Consult the stormwater management manual for the local regulatory jurisdiction and the Washington State Stormwater Management Manual for Western Washington (SWMMWW) regarding possible regulatory restrictions for:

Predeveloped Forest Cover - There are restrictions concerning the designation of the predeveloped land use as anything other than forest (SWMMWW Volume I, Minimum Requirement 7),

Post Developed Forest Or Pasture Cover - Assurances are required when designating an area as forest or pasture for the postdevelopment state to ensure that the area will not be disturbed in the future (SWMMWW Volume III, Appendix B),

Off-Site Run-On To Project - There are limits to offsite inflow discharging to a stormwater detention facility (SWMMWW Volume III, Appendix B),

On-Site Stormwater Bypass - There are restrictions to the size of development area from which stormwater runoff may bypass a detention facility (SWMMWW Volume III, Appendix B).

Mapping of soil types by the Soil Conservation Service (SCS) is the most common source of soil/geologic information used in hydrologic analyses for stormwater facility design. Each soil type defined by the SCS has been classified into one of four hydrologic soil groups; A, B, C, and D. The *Stormwater Management Manual for Western Washington*<sup>9</sup> relates SCS hydrologic soil groups to HSPF soil/geologic groups as shown in Table 5.1

**Table 5.1 – Relationship between SCS and HSPF Soil Groups**

SCS Hydrologic Soil Group	MGSFlood/HSPF Soil/Geologic Group
A/B	Outwash
C	Till
D	Wetland

Note: The surface area of the pond must be included under the land use for the subbasin because precipitation is not applied to the pond surface by the program. This can be accomplished by adding impervious surface equal to the maximum pond surface area under the Subbasin Definitions window.

## 5.2 By-pass Areas

Local topographic constraints often make it impractical to direct all runoff from developed areas to a detention facility. If a portion of the developed watershed bypasses the tributary node, then a secondary by-pass node can be specified (Node 2 in Figure 5.1). This feature is useful for allowing a portion of the developed site to bypass the stormwater detention pond and the downstream node is used as the point of compliance. Acreage for each cover/soil type combination that bypasses the collection node is specified and runoff from this part of the development is collected at a user-specified by-pass node.

## 5.3 Node Connections

Node connections allow the user to connect runoff from multiple subbasins at a single point or connect bypass flows to a location downstream of the stormwater pond. In the example shown in Figure 5.2, Subbasins 1, 2, and 3 are connected to Nodes 1, 2, and 3 respectively,

which represent the runoff from each subbasin. The runoff from each subbasin is routed to Node 4 with each Link defined as an open channel in this case. Node 4 represents the inflow to a stormwater pond. Two acres of Subbasin 3 bypasses the pond to Node 5. Runoff from Node 4 will be used as input for sizing the stormwater detention pond with Node 5 used to save routed flows from the pond. The bypassed flows from Subbasin 3 will be added to the pond outflow during routing, and Node 5 will be the point of compliance. Following routing computations, flows stored at Node 5 will be the sum of the routed flows and the bypassed flows from Subbasin 3. Note that no routing of runoff from the bypass area occurs. Flows from bypassed areas are added to the downstream node without routing.

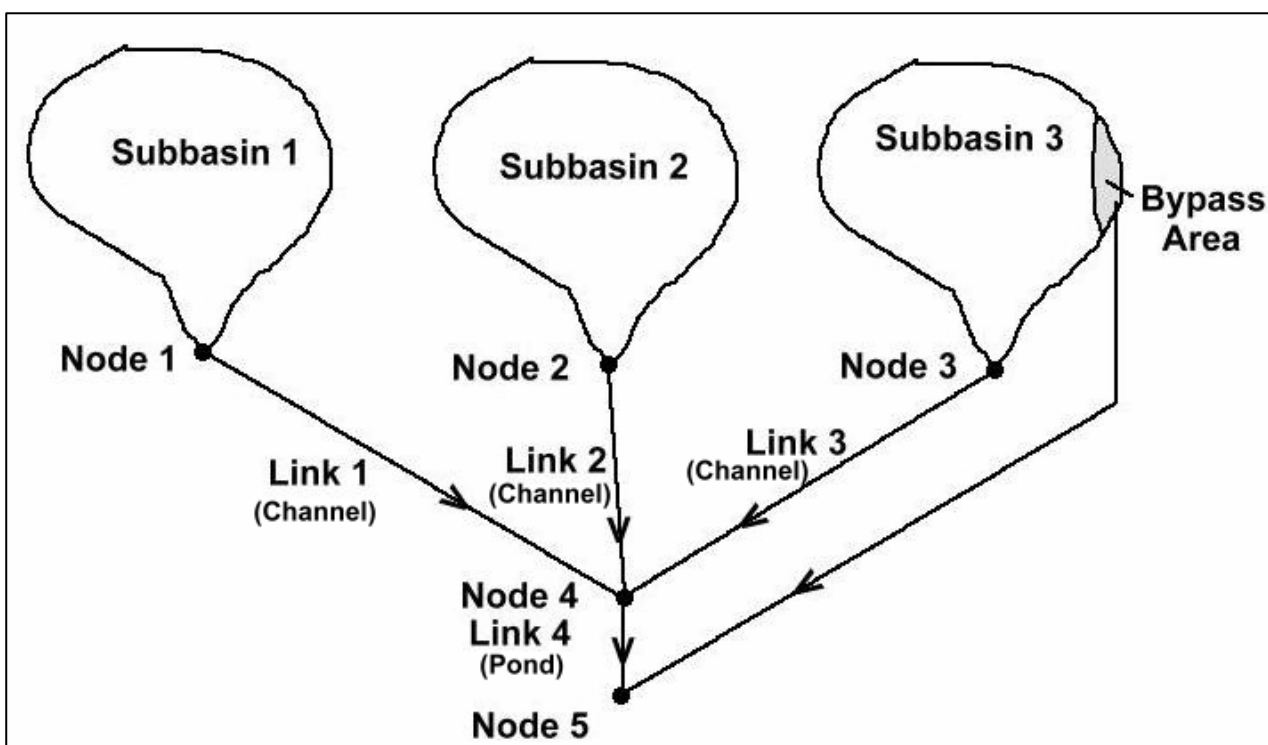


Figure 5.2 – Subbasin, Node, and Link Delineation, Multiple Subbasin Example with Bypass

## 5.4 Lateral Flow Connections

MGSFlood simulates runoff from pervious areas as three components; surface overland flow, interflow, and groundwater flow. The Lateral Flow Connection Option screen can be used to define which runoff components are connected to the receiving node. By default, surface and interflow are connected to the subbasin receiving node with the groundwater component omitted. In cases where high groundwater levels results in discharge to the drainage system, the groundwater component may be included in the runoff calculations.

Runoff from upslope areas may also be connected to downslope pervious areas to simulate surface runoff dispersion and infiltration. Figure 5.3 shows runoff from an area of impervious surface that is dispersed over a pervious area before reaching the receiving stream. The infiltration of the impervious runoff would be simulated in the model by connecting the impervious surface runoff to the downslope Lateral Flow PerInd. Each subbasin has two separate Lateral Flow PerInds that can be defined to simulate flow dispersion. Upslope surface flow, interflow, and groundwater may be connected to the downslope surface, interflow, or groundwater flow components. Using HSPF nomenclature, the upslope components represent SURO, IFWO, and AGWO which may be connected to the downslope SURLI, IFWLI, or AGWLI.

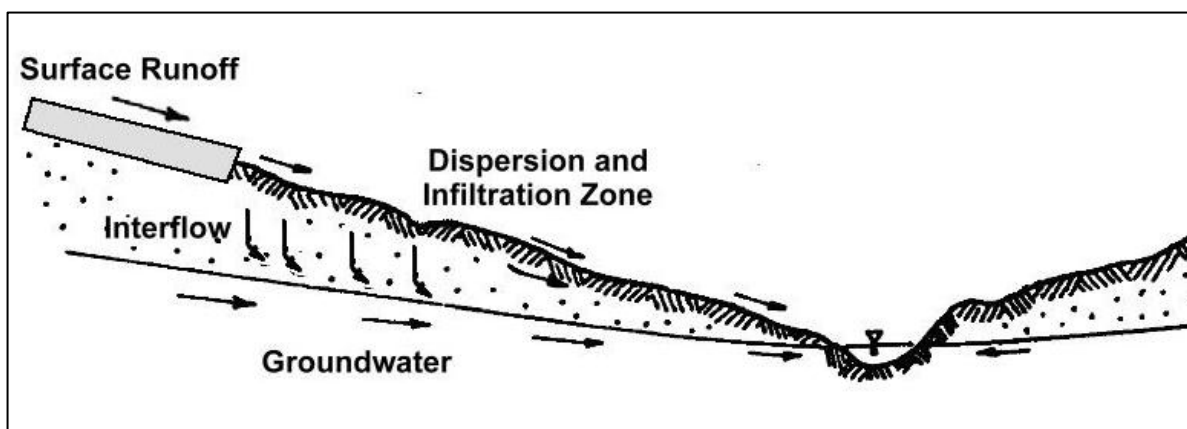


Figure 5.3 – Surface Runoff Dispersion and Infiltration from Upslope Impervious Surface over Downslope Pervious Surface

Figure 5.4 shows an example schematic where runoff from impervious surface and surface and interflow from till/grass are connected to a downslope pervious area. The upslope surface components are connected to the downslope surface component, the upslope interflow component is connected to the downslope interflow component, and the downslope surface and interflow components are connected to the receiving node. Figure 5.5 shows the input screen in the MGSFlood model for this example. Note that the runoff parameters for Lat 1 (the downslope PerInd) are defined in the list box in the lower right of the screen.

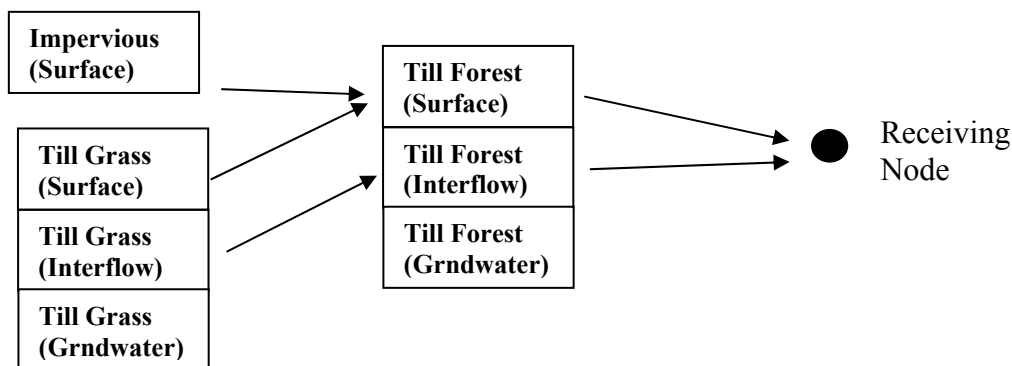


Figure 5.4 – Example Flow Dispersion Schematic, Upslope Impervious and Grass Routed to Downslope Till Forest Area (Note, only surface and interflow connected in this example)

**Lateral Flow/High Groundwater Definitions**

**Lateral Flow**    High Groundwater 1    High Groundwater 2

**Lateral Flow Connections (Postdeveloped)**

Upstream Segment	Connect to:	Define Upstream Active Components			Lateral Flow Runoff Parameters
		Surface to:	Interflow to:	Groundwater to:	
Till Forest	Node	Node	Node		
Till Pasture	Node	Node	Node		
Till Grass	Lat 1	Surface	Interflow		
Outwash Forest	Node	Node	Node		
Outwash Pasture	Node	Node	Node		
Outwash Grass	Node	Node	Node		
Wetland	Node	Node	Node		
User1	Node	Node	Node		
User2	Node	Node	Node		
Impervious	Lat 1	Surface			
<b>Lateral 1</b>	Node	Node	Node		Till Forest
Lateral 2	Node	Node	Node		Till Forest

View Schematic

Ok    Cancel

**Figure 5.5 – Upslope Impervious and Till Grass Connected to Downslope Till Forest,  
Surface and Interflow Only  
Downslope Till Forest Surface and Interflow are Connected to the Receiving Node**



## 6 Network Definition

Nodes are used to collect runoff from the tributary area for a given subbasin and from the nodes of upstream subbasins. Links are used to connect one node to another and may be defined as one of the following:

1. Copy – Copies discharge from upstream node and adds it to discharge at the downstream node without routing,
2. Structure – Includes detention and infiltration ponds, and sand filters,
3. Channel – Performs routing in open channels,
4. Infiltration Trench – Performs routing through infiltration trenches,
5. Rating Table – User defined stage storage discharge table,
6. Flow Splitter – Splits a fraction of the discharge from one node to another.

Currently, the program is configured to accommodate a maximum of 10 nodes with 9 links. Separate predeveloped and postdeveloped network configurations are defined to accommodate the sizing of stormwater treatment facilities in the post-developed condition. Information for each type of Link is discussed in the following Sections.

## 7 Copy Link

The copy link copies timeseries from the upstream node and adds it to the downstream node. Hydrographs are transferred to the downstream node without attenuation or lagging. The copy link is appropriate for small watersheds where there is little attenuation of the flood hydrograph from one node to the next. If the conveyance channel is long with large overbank storage, then the link should be defined as an open channel. As a general rule, channel routing may be neglected for watersheds smaller than about ½ square mile (320 acres) and the link may be defined using the copy routine.



## 8 Structure Link

Structure links are used to define stormwater ponds, infiltration ponds, and sand filters. Pond optimization information for post-development condition ponds is also input on the structure link input screens.

A variety of hydraulic devices can be included in the design of stormwater treatment facilities. Devices attached to the riser structure include; circular orifices, circular orifices under backwater influence, rectangular orifices, rectangular weirs, V-notch weirs, and proportional weirs. In addition, the riser structure can also be defined with an open top to function as an overflow weir, or the top may be capped. Any combination of up to six devices plus the riser structure and a sand filter can be included for each structure. A trapezoidal broad crested weir may also be specified to function as an emergency overflow.

The following sections describe the input for Structure Links.

### 8.1 Pond/Vault Geometry Input

Two options are available for specifying pond or vault geometry. The first assumes a prismatic geometry with pond length, width, depth, and side slopes as shown in Figure 8.1.

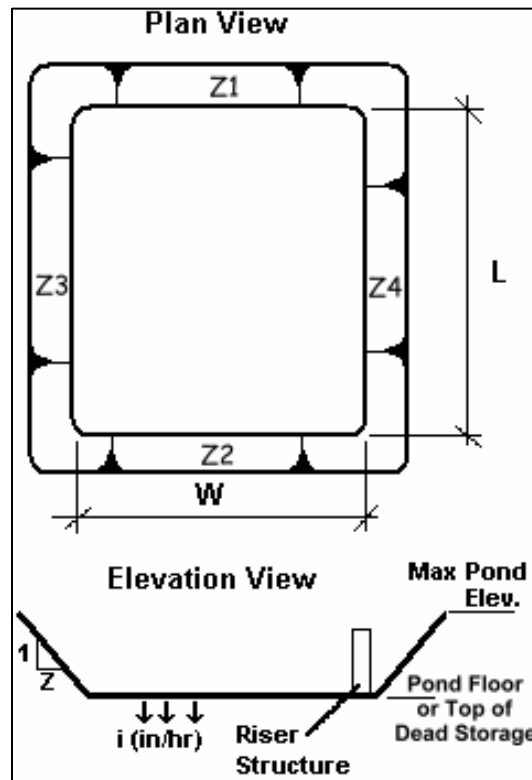


Figure 8.1 – Pond Geometry Definition for Prismatic Ponds

where:

$L$  – is the pond length in feet,

$W$  – is the pond width in feet,

$Z1, Z2, Z3, Z4$  - are side slopes for each side of the pond where  $Z$  is the number of feet in the horizontal plane for every foot of rise,

*Pond Floor Elevation* – Represents the bottom of the *live* pond storage. Live storage is defined as the storage used to detain stormwater runoff and eventually flows through the outlet structure. Dead storage is retained in the pond below the elevation of the outlet structure. The pond floor elevation should be input if the pond is not a combined wet pond. If the pond is a combined wet pond, then enter the elevation of the top of the dead storage, i.e. the elevation where water begins to discharge from the pond

*Riser Crest Elevation* – The elevation at which water begins to flow into the overflow riser. The maximum flood recurrence interval detained by the pond generally corresponds with this elevation (or slightly above this elevation). For example, the Ecology flow duration standard requires control of the flow duration between  $\frac{1}{2}$  of the 2-year and the 50-year recurrence interval. Water will begin to spill into the riser structure near the 50-year recurrence interval. It is acceptable for water to spill into the riser structure for floods smaller than the 50-year provided that the flow duration standard is met.

*Max Pond Elevation* – Is the maximum elevation used in pond routing calculations and typically extends above the riser crest elevation a sufficient distance to accommodate large floods or to allow for flood passage if one or more of the lower level outlets become blocked. The required maximum pond elevation depends on the design standards of the local jurisdiction.

The automatic pond sizing routine (optimizer) in MGSFlood determines the riser diameter and maximum pond elevation so that the 100-year peak inflow will pass through the riser structure assuming the lower level outlets are blocked. The user is advised to check the maximum pond elevation returned by the optimizer with the design standards of the local jurisdiction including any freeboard requirements.

If a vault is to be analyzed, then side slopes ( $Z1, Z2, Z3, Z4$ ) of zero are input denoting vertical sides. The pond volume for elevations ranging from the floor to one foot above the maximum pond elevation is computed according to this geometry.

The second method for specifying pond geometry is with a user defined elevation-volume table as shown in Figure 8.2. This is useful for specifying the geometry of irregularly shaped ponds. The elevation-volume relationship can be computed using a spreadsheet program and pasted into the form using the Windows Clipboard utility.

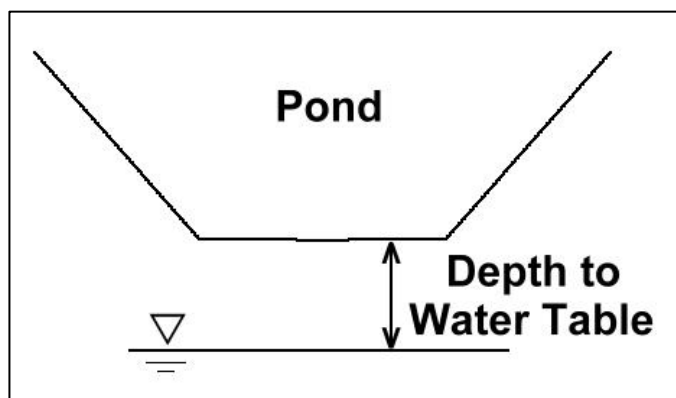
Note: Precipitation falling on the surface of the detention pond is not automatically computed by MGSFlood. This approach was taken to allow use of both ponds and vaults. The difference being ponds are open to collection of precipitation, and vaults are closed to precipitation input. To include precipitation on the pond surface in the computations, the surface area of the pond must be included under the land use for the subbasin where the pond resides. This can be accomplished by adding impervious surface equal to the maximum pond surface area under the *Subbasin Definitions* window for the sub-basin where the pond resides. A simple approach to get an initial estimate of the pond surface area would be to run the *Quick Optimization routine* after the tributary subbasins have been defined.

Row	Elev (ft)	Surf Area (sf)	Volume (cu-ft)
1	100.00	0.0	0.00
2	101.00	2500.0	2500.0
3	102.00	5000.0	8000
4			
5			
6			
7			
8			
9			
10			
11			
12			
13			
14			
15			

**Figure 8.2 – Pond Elevation-Volume Entry Screen  
(Useful for Irregularly Shaped Ponds)**

## 8.2 Pond Infiltration

MGSFlood simulates infiltration using an approach developed by Massmann<sup>30</sup> based on field observations of infiltration ponds in western Washington. This infiltration approach accounts for the side slope geometry of the pond, pond aspect (length to width ratio), the proximity of the pond to the regional groundwater table, and the potential for soil clogging and fouling. Inputs include; Soil Hydraulic Conductivity (inches/hour), Depth to the Regional Water Table (ft) (Figure 8.3), whether bio-fouling potential is low, and whether average or better maintenance is performed. Infiltrated moisture is lost from the system and does not contribute to the discharge rate through the riser or orifices.



**Figure 8.3 – Infiltration Pond Depth to Water Table  
(Accounts for Groundwater Mounding Beneath Pond)**

Soil Hydraulic Conductivity (in/hr) – Is the saturated hydraulic conductivity of the soil beneath the pond in inches per hour according to Darcy's Equation. It can be estimated using regression equations that use grain size distribution as input (Massmann<sup>30</sup>) or from literature (e.g. Freeze and Cherry<sup>31</sup>, Fetter<sup>32</sup>).

Depth to Regional Groundwater Table (ft) – Represents the depth from the bottom of the pond to the regional groundwater table or the first low-permeability layer. For shallow groundwater sites, groundwater mounding reduces the hydraulic gradient and the infiltration rate is significantly less than the saturated hydraulic conductivity. For deep groundwater sites where the effects of mounding will be small, the gradient will not typically be reduced by infiltration from the facility. Increasing the depth to groundwater greater than 100 feet ceases to have an influence on pond infiltration according to this approach.

Bio-fouling Potential – Bio-fouling occurs from organic material blanketing the soil surface and reducing the infiltration rate. Bio-fouling is more likely to occur if the pond is located beneath trees and other vegetation or in shaded locations.

Maintenance – Siltation is more likely to occur if there is not sufficient pre-treatment of the storm water or in locations where the drainage basin is prone to erosion because of recent land disturbances or steep slopes. The user should

consider the potential for siltation of the infiltration pond and the maintenance program when determining the effects of maintenance on pond infiltration performance.

### 8.3 Circular Orifice

Orifices<sup>3,6</sup> can be defined as oriented in either the vertical or the horizontal plane. The discharge for orifices oriented horizontally, or fully submerged orifices in the vertical plane (Figure 8.4) are computed using Equation 8.1.

$$Q = C_d A \sqrt{2gH} \quad 8.1$$

Where:  $Q$  is the discharge at a given pond water surface elevation,  
 $C_d$  is a coefficient of discharge (0.61 without elbow, 0.58 with elbow),  
 $A$  is the orifice area,  
 $g$  is the acceleration due to gravity, and  
 $H$  is the head, as measured between the pond water surface elevation and the water surface elevation at the orifice outlet.

For orifices mounted in the vertical plane and not subject to backwater from the outlet conduit (Figure 8.4), head ( $H_m$ ) is measured as the difference between the water surface elevation in the pond and the elevation of the centroid of the orifice.

For orifices mounted in the horizontal plane and not subject to backwater from the outlet conduit (Figure 8.4), head ( $H_t$ ) is measured as the difference between the water surface elevation in the pond and the water surface elevation at the outlet from the orifice.

Note when specifying the elevation of the lowest orifice (such as the bottom orifice in Figure 8.4) the controlling elevation that governs the head on the orifice must be identified. The controlling elevation may be the invert elevation of the orifice, the centerline elevation of the orifice, the invert elevation of the outlet conduit, or the hydraulic grade line in the outlet pipe.

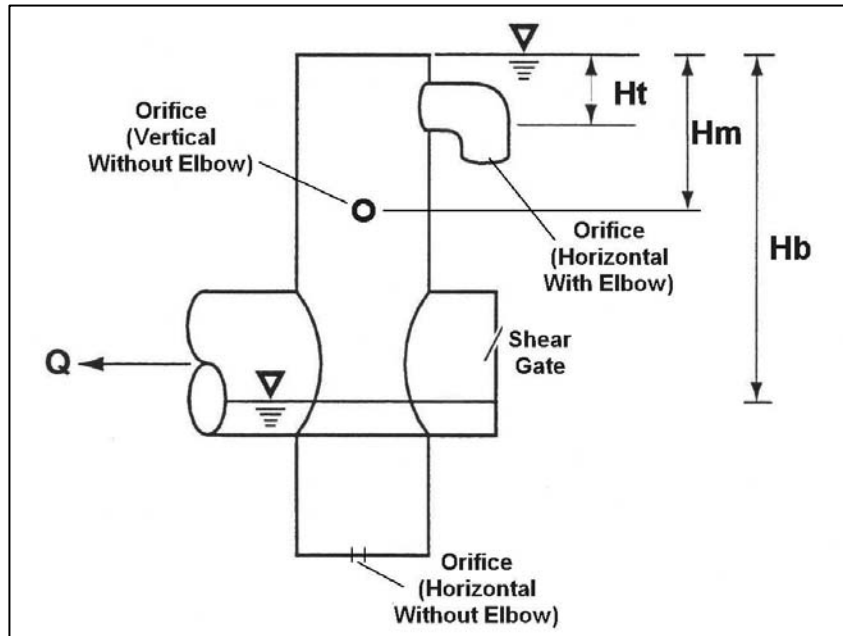


Figure 8.4 – Riser Structure Schematic for Three Possible Orifice Configurations

For orifices oriented in the vertical plane, where the water surface in the pond results in the orifice flowing partly full, discharge is computed based on critical depth occurring at the orifice face. The transition from flowing partly full to orifice flow occurs when the head ( $H_i$ ) is near 110% of the orifice diameter. The governing discharge relationships for this situation (Equations 8.2a,b,c) are based on critical depth occurring in a circular section at the orifice face<sup>4</sup>.

$$Q = A_c \sqrt{\frac{gD_c}{\alpha}} \quad 8.2a$$

$$\text{and } D_c = A_c/T \quad 8.2b$$

$$\text{and } H_i = Y_c + D_c/2 \quad 8.2c$$

Where:  $Q$  is the discharge at a given pond water surface elevation,  
 $Y_c$  is critical depth at the face of the circular orifice,  
 $A_c$  is the cross-sectional area of flow at critical depth,  
 $T$  is the top width of flow at the orifice opening for critical depth,  
 $D_c$  is the hydraulic depth,  
 $H_i$  is the head on the orifice, as measured from the water surface elevation of the pond to the invert elevation of the orifice,  
 $g$  is the acceleration due to gravity, and  
 $\alpha$  is a type of discharge coefficient (1.00)

## 8.4 Circular Orifice with Tailwater

Discharge for the lowest circular orifice can be computed with or without tailwater conditions due to downstream controls such as pipe networks. If tailwater conditions are present, an elevation-discharge rating table must be included that describes the tailwater condition downstream. A minimum of four elevation-discharge pairs are needed to define the tailwater rating table. The program uses an iterative procedure whereby the discharge computed using Equation 8.1 is based on the difference between the pond water surface elevation and the water surface elevation in the outlet conduit/riser section ( $H_b$ , Figure 8.4), and the computed discharge matches the discharge and tailwater elevation obtained from the rating table defining the downstream conditions.

Some possible applications for the tailwater routine might include:

- a) Tailwater from a lake;
- b) Tailwater from another stormwater pond;
- c) Tailwater from high groundwater level that causes backwater against the outfall of the outlet conduit;
- d) Tailwater from high tide or other tidal influence;
- e) Tailwater from floodwaters from a receiving stream or overbank area of a floodplain;
- f) Tailwater from concurrent discharges where the pond outlet connects into a closed stormwater system.

Cases a) and c): – tailwater may be essentially fixed with a very small change in tailwater elevation for various discharges from stormwater pond. The user would enter a constant tail water elevation for each entry in the elevation-discharge table. Discharge values would then be entered that covered the full range of possible discharges for the pond.

Cases e) and f): – if the receiving systems are sufficiently complex and difficult to analyze, an analysis approach would be to assume the frequency of floods discharging from the pond are similar to the receiving system. That is, the pond discharges at a 10-year recurrence interval at the time the receiving system is experiencing a 10-year flood. This assumption would allow determination of a tailwater level for the receiving system (floodplain analysis) and obtaining the corresponding 10-year flood discharge from MGSFlood for the stormwater pond.

### Design Steps For Tailwater Situations

1. Design the outlet structure for the stormwater pond for the case of no tailwater to provide an initial estimate of the configuration of the outlet structure and pond. Note the maximum discharge from the pond. This provides the range of possible discharges from the pond (0 cfs to maximum discharge);
2. Review the flood-frequency curve for pond discharges (MGSFlood) to provide information on the frequency of occurrence of various discharges throughout the range of possible discharges. This information may be helpful if the tailwater conditions vary based on the magnitude of the concurrent flood event in the receiving system. This may be true for cases a), b), e) and f) above.

3. Determine the range of reasonable tailwater elevations through analysis, judgment, and/or policy for the range of possible discharges from the stormwater pond. Tailwater conditions may be independent of discharge magnitude from the stormwater pond (Case d), or they may be related through seasonality (Cases a, c), or they may be related by concurrent flood events (Cases a, b, e, f). Provide a minimum of four data pairs for tailwater elevation and corresponding discharge (Steps 1 and 2) that reflect the operation of the “system” that is causing the tailwater condition. The tailwater elevations must be distinct values, even if only slightly different from one-another for the range of possible discharges.
4. Rerun the problem with MGSFlood using the tailwater elevation-discharge rating curve obtained from Step 3 and note how the range of possible discharges has changed from the no tailwater case (Step 1).
5. If the revised tailwater elevation-discharge relationship is significantly different (based on solution from Step 4), then use the revised tailwater elevation-discharge relationship and rerun the problem again. Continue iterating until the proposed tailwater elevation-discharge relationship is consistent with that obtained for the solution of the pond configuration and the range of possible discharges from the pond.

## 8.5 Rectangular Orifice/Slot

A rectangular orifice<sup>3,6</sup> functions as an orifice when submerged at the orifice entrance, or as a rectangular sharp crested weir when partially submerged at the orifice entrance. This approach is also used for rectangular orifices (vertical slots) cut in the side of the riser to the riser crest.

It is assumed that a rectangular orifice would be mounted near mid-height on the outlet structure and would not be subject to tailwater conditions. Equation 8.1 is used in the calculation of discharge from a rectangular orifice if the orifice is submerged at the orifice entrance. If the orifice is constructed in the vertical plane, then it is treated as a rectangular sharp crested weir according to Equations 8.5a and 8.5b for low heads. At higher heads, the rectangular opening behaves as an orifice and Equation 8.1 is used. The transition from weir flow to orifice flow occurs when the depth above the slot invert reaches the value as defined in Equation 8.3. Note that the height of the orifice can be any size up to the top of the riser.

$$H_t = 1.60 (1.0 - B/L) + 1.08 \quad 8.3$$

Where:  $H_t$  is the depth above the orifice bottom where the discharge changes from weir flow to orifice flow  
 $B$  is the effective weir length as defined in Equation 8.4b  
 $L$  is the weir length.

## 8.6 V-Notch Sharp Crested Weir

The V-Notch weir is assumed to be constructed in the side of the riser structure with the top of the notch intersecting the riser crest (Figure 8.5a) and would not be subject to tailwater conditions. Discharge from the V-Notch weir<sup>3,6</sup> is computed according to Equation 8.4, where the weir coefficient ( $C_w$ ) can be obtained from Figure 8.5b as described by Daugherty and Franzini<sup>6</sup>. Limited test data are available for V-notch weirs with narrow openings (small  $\theta$ , less than  $10^\circ$ ), users should be aware of greater uncertainty in the discharge coefficients for narrow openings.

$$Q = C_w T \tan\left(\frac{\theta}{2}\right) H^{2.5} \quad 8.4$$

Where:  $Q$  is the discharge at a given pond water surface elevation,  
 $C_w$  is a weir coefficient of discharge (Figure 8.5b),  
 $\theta$  is interior angle of the V notch in degrees, and  
 $H$  is the head above the weir invert.

When the water surface elevation in the pond exceeds the riser lip elevation and results in a 10 percent or more increase in the head ( $H$ ) on the weir over the head associated with the riser lip elevation, the entrance to the V-notch weir becomes submerged and the orifice equation (Equation 6.1) is used to compute the discharge through the V-Notch. In this situation, the head on the orifice is measured from the pond water surface elevation to the centroid of the V-notch opening.

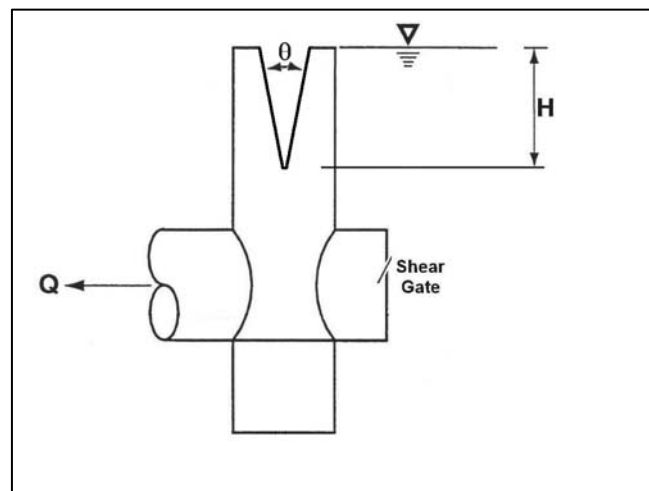


Figure 8.5a – V-Notch Weir

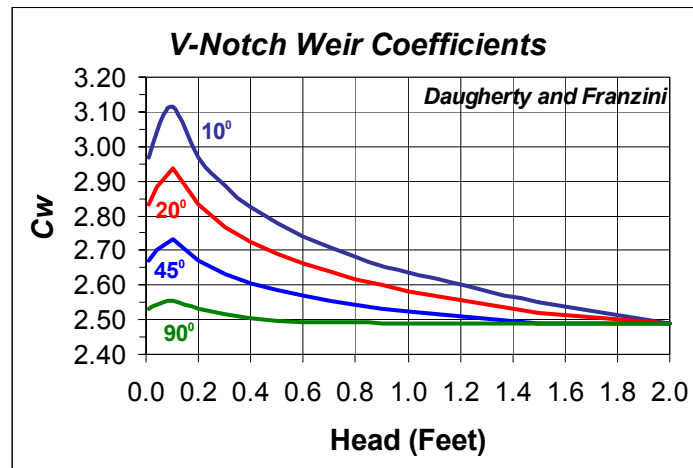


Figure 8.5b – Weir Coefficients for Various Opening Angles of V-Notch Weirs

## 8.7 Rectangular Sharp Crested Weir

The rectangular sharp crested weir<sup>3,6</sup> is assumed to be constructed in the side of the riser structure with the top of weir intersecting the riser crest (Figure 8.6) and would not be subject to tailwater conditions. Discharge from the weir is computed according to Equations 8.5a,b where  $L$  is the weir length and the weir coefficient ( $C_w$ ) is 3.33. For narrow sharp crested weirs that function as a slot, contraction of the flow at the sides of the weir yields an effective weir length ( $B$ ) equal to 80% of the constructed weir length ( $L$ ).

$$Q = C_w B H^{1.5} \quad 8.5a$$

$$B = L(1.0 - 0.2H) \quad \text{and} \quad B \geq 0.8L \quad 8.5b$$

Where:  $Q$  is the discharge at a given pond water surface elevation,  
 $C_w$  is a weir coefficient (3.33),  
 $B$  is the effective weir length, which is a function of the weir length ( $L$ ) and the head on the weir (Equation 8.5b), and  
 $H$  is the head as measured above the weir invert.

If the weir opening is narrow relative to the height, then it will behave as a weir at low heads and an orifice at higher heads. The transition from rectangular sharp crested weir (Equations 8.5a and 8.5b) to orifice (Equation 8.1) occurs when the depth above the weir invert reaches the value as defined in Equation 8.3. In this situation, the head on the orifice is measured from the pond water surface elevation to the centroid of the weir opening.

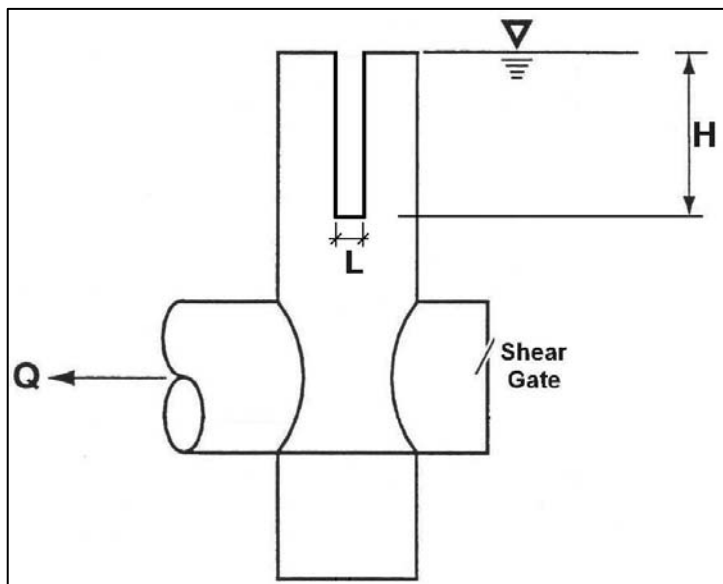


Figure 8.6 – Rectangular Sharp Crested (Slot) Weir

## 8.8 Proportional Weir

The proportional weir is assumed to be constructed in the side of the riser structure with the top of weir intersecting the riser crest (Figure 8.7) and would not be subject to tailwater conditions. The proportional weir has curved sides such that the discharge through the weir varies linearly with head<sup>16</sup> (Equation 8.6).

$$Q = C_d L \sqrt{2gb} (H - b/3) \quad 8.6$$

Where:  $Q$  is the discharge at a given pond water surface elevation,  
 $C_d$  is a coefficient of discharge (0.60),  
 $L$  is the weir length at the base,  
 $g$  is the acceleration due to gravity,  
 $H$  is the head above the weir invert, and  
 $b$  is the height of the vertical portion of the weir sidewall.

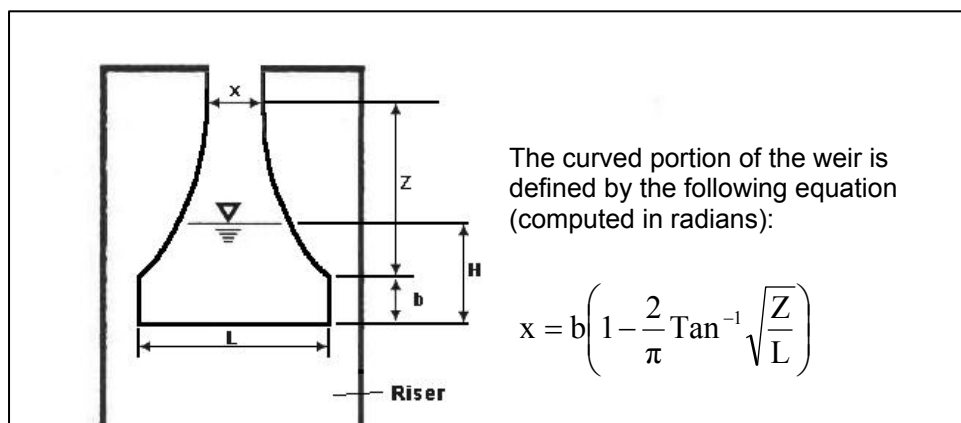


Figure 8.7 – Proportional Weir

When the water surface elevation in the pond exceeds the riser crest elevation, the entrance to the weir becomes submerged and the orifice equation (Equation 8.1) is used to compute the discharge through the weir opening. In this situation, the head on the orifice is measured from the pond water surface elevation to the centroid of the weir opening.

## 8.9 Trapezoidal Broad Crested Weir

The trapezoidal broad crested weir<sup>3,4,6</sup> is commonly used as an emergency overflow structure and is assumed to be constructed on the pond/vault rim and does not interact with the riser structure. Discharge from the broad crested weir is assumed to be conveyed to the discharge point for the pond and is added to the discharge from any other structures associated with the pond.

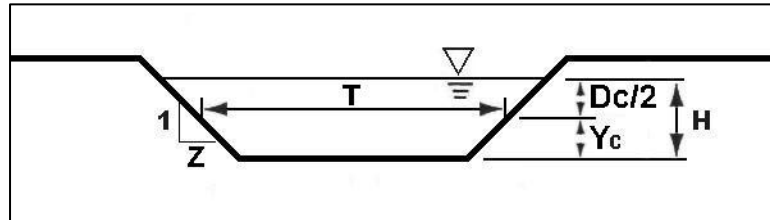
The governing discharge relationships (Equations 8.7a,b,c) for the broad crested weir are based on critical depth occurring on the weir crest<sup>4</sup> (Figure 8.8).

$$Q = A_c \sqrt{\frac{gD_c}{\alpha}} \quad 8.7a$$

$$\text{and } D_c = A_c/T \quad 8.7b$$

$$\text{and } H = Y_c + D_c/2 \quad 8.7c$$

Where:  $Q$  is the discharge at a given pond water surface elevation,  
 $Y_c$  is critical depth on the weir,  
 $A_c$  is the area of discharge at critical depth,  
 $T$  is the top width of flow at the weir opening for critical depth,  
 $D_c$  is the hydraulic depth at the weir opening,  
 $H$  is the head on the weir, as measured from the water surface elevation of the pond to the invert elevation of the weir,  
 $g$  is the acceleration due to gravity, and  
 $\alpha$  is a type of discharge coefficient (1.20)



**Figure 8.8 – Broad Crested Trapezoidal Weir**

The critical depth-equations can be reformulated into the general form of the weir equation (Equation 8.5a) for a broad-crested weir with a rectangular cross-section. In this format, an alpha value ( $\alpha$ ) of 1.00 yields a weir coefficient ( $C_w$ ) of 3.089, the maximum value for rectangular broad-crested weirs. Use of an alpha value ( $\alpha$ ) of 1.20 results in a weir coefficient of 2.82, which is applicable for broad-crested weirs operating at shallow depths. This would represent typical conditions for a broad-crested weir operating as an emergency spillway for a detention pond.

## 8.10 Riser Structures

A single riser structure can be specified for each pond analyzed. The riser can be either circular or rectangular in cross section with the top either closed (capped) or open. When the top is open, discharge is allowed to occur over the riser crest effectively functioning as an overflow spillway. If the riser top is open, discharge over the riser rim is computed according to Equations 8.8a,b.

Hydraulic structures that intersect the riser crest, such as V-notch or rectangular sharp crested weirs, are accounted for by entering a *common length*. This ensures that the discharge from the hydraulic structure is not double counted when flow passes over the riser crest. For the example shown in Figure 8.9, the 1.5-foot wide rectangular sharp crested weir intersects the riser crest. A value of 1.5 feet would be input as the *common length* under the Riser Structure Parameters.

For narrow devices that intersect the riser crest, there is little difference between the slot width (chord length on circle) and the arc length. Thus, the slot width can be entered for the common length. For structures that are wide relative to the diameter of the riser, the arc length should be computed and entered for the common length (Figure 8.10).

$$Q = C_w B H^{1.5} \quad 8.8a$$

$$B = L - \text{common length} \quad 8.8b$$

Where:  $Q$  is the discharge at a given pond water surface elevation,  
 $C_w$  is a weir coefficient that is initially 3.33, and decreases with increasing head ( $H$ ) on the weir<sup>28,29</sup>,  
 $L$  is the weir length as measured along the circumference of the riser top,  
 $B$  is the effective weir length, which is a function of the weir length ( $L$ ) and reduced by any common length with other discharge devices according to Equation 8.8b, and  
 $H$  is the head as measured above the riser lip.

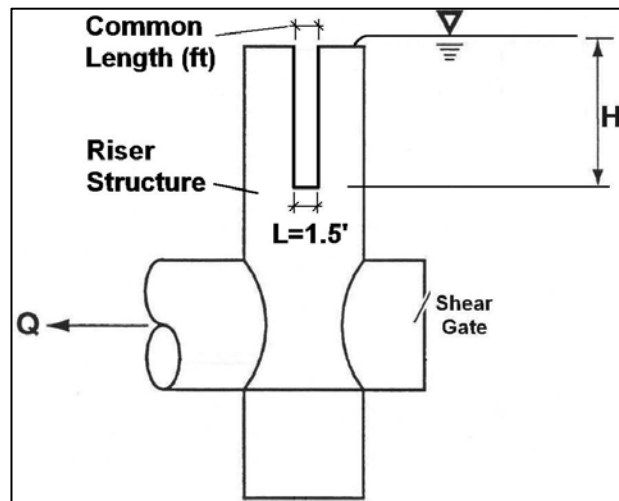
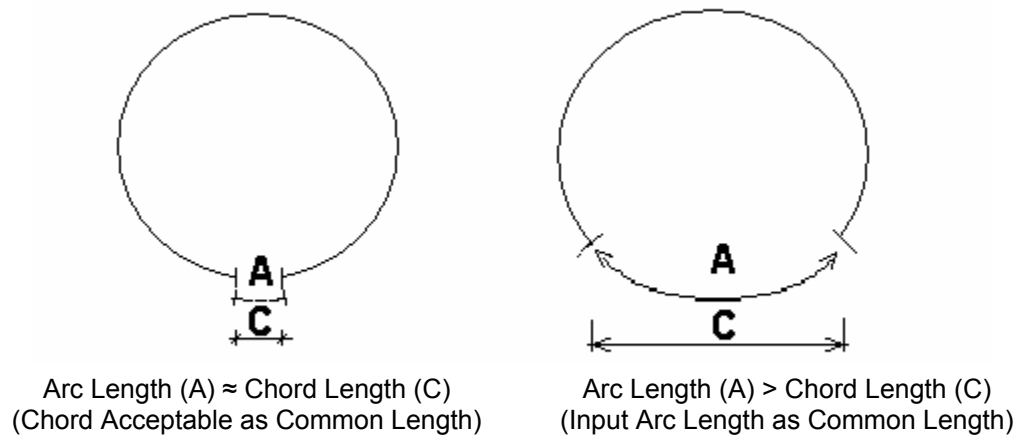


Figure 8.9 – Riser Structure



**Figure 8.10 – Plan View of Circular Riser Showing Common Length for Narrow and Wide Weir**

### 8.11 Sand Filter

A sand filter functions much like an infiltration pond except that instead of infiltrating into native soils, stormwater filters through a constructed sand bed with an underdrain system. Unlike infiltration from the pond bottom, the underdrain system is connected to the stream network and any discharge from the sand filter is added to the discharge from the outlet structure associated with the pond. The sand filter offers a mechanism for release of very small quantities of discharge as an alternative to a very small low-level circular orifice that is susceptible to debris blockage. A sand filter also removes pollutants by filtration. As stormwater passes through the sand, pollutants are trapped in the interstices between the sand grains.

It is assumed that discharges via a sand filter would be minimal relative to other discharge devices, and there would be no tailwater conditions present when the pond depth is above the sand filter but below any intermediate hydraulic device.

The discharge rate through a sand filter is computed using Darcy's law<sup>6</sup> (Equations 8.9a,b).

$$Q = KiA \quad 8.9a$$

$$i = H/L \quad 8.9b$$

Where:  $Q$  is the discharge through the filter for a given pond water surface elevation,  
 $K$  is the saturated hydraulic conductivity (permeability)  
 $i$  is the hydraulic gradient through the filter  
 $H$  is the head on the filter at a given pond water surface elevation  
 $L$  is the filter thickness, and  
 $A$  is the surface area of the filter perpendicular to the direction of flow.

The percentage of pond inflow that passes through the sand filter is computed by the program and listed in the project report. Sizing the sand filter area is a trial and error procedure whereby different filter surface areas are tried; flows are routed through the facility and the percentage of runoff treated by the filter is noted from the project report. The process is repeated until the required level of treatment is achieved.

## 8.12 Automatic Pond and Outlet Works Sizing Routine

The proprietary version of MGSFlood includes routines for computing pond hydraulics and automatically sizing the pond and outlet works to meet the Washington State Department of Ecology Flow Duration Standard<sup>9</sup>. Designing stormwater ponds to this standard is a laborious, iterative process whereby the runoff timeseries (typically 40-years or more) is routed through the pond, flow-duration statistics computed and then compared with pre-developed flow-duration statistics. The automatic pond sizing routine performs this pond design procedure automatically.

The automatic pond sizing optimization routine in the MGSFlood will determine the pond size and outlet configuration for two pond types; a detention pond with minor infiltration and an infiltration pond (the routine will also automatically size infiltration trenches, see Section 10.3). The characteristics of these two pond types are listed in Table 8.1

<b>Table 8.1 – Characteristics of Detention and Infiltration Ponds Sized using Optimization Routine</b>		
<b>Characteristic</b>	<b>Detention Pond</b>	<b>Infiltration Pond</b>
Pond Configuration	Riser Structure with Low Level Circular Orifice and Vertical Rectangular Upper Orifice	Overflow Riser Only
Valid Infiltration Rates*	0.00 – 0.10 inches/hour	0.05-50 inches/hour
Optimization Levels	Quick or Full	Quick Only

\* Note: Infiltration occurs through the pond bottom only, not including the side slopes.

The pond sizing optimization routine uses general input about the pond geometry including;

- Pond length to width ratio,
- Pond side slope,
- Pond floor elevation,
- Riser crest elevation, and
- Pond infiltration information.

The pond sizing routine uses the information listed above to establish the geometric relationships for the pond configuration. The program establishes a parameter space of possible solutions by varying the pond bottom area, and sizes and elevations of hydraulic devices for the outlet structure. The program then routes the developed runoff timeseries through the pond and seeks to find a solution that provides the minimum pond size to meet the duration design standard.

The standard outlet configuration used for detention ponds consists of a circular low-level orifice and a vertical rectangular orifice (slot). If a different outlet configuration is desired,

the volume-discharge characteristics of the desired configuration can be set to match the volume-discharge characteristics returned by the program for the orifice/slot weir configuration. The low-level circular orifice is assumed to be free of tailwater effects. If tailwater conditions are present, first use optimization to determine the pond configuration without tailwater. Then, include the tailwater rating table and manually adjust the pond configuration to meet the duration design criteria.

There are a wide variety of combinations of hydraulic devices, device sizes and invert heights, and pond configurations that can be used to match the flow duration standard. However, it is difficult to find a pond configuration that minimizes the pond volume and meets the duration standard using a manual trial and error approach. The automatic pond sizing routine searches the parameter space of possible solutions and seeks to find the minimum pond size to meet the flow duration standard.

The following steps describe the pond design process using the Hydraulic Structures, Optimization routine.

Step 1. Input land use on the *Watershed Layout* tab and move to the *Compute Runoff* tab and compute runoff.

Step 2. Select the type of pond to be sized, either detention or infiltration. Enter the general pond geometry on the *Optimization* tab of the *Hydraulic Structures* Input Screen. The geometry consists of the pond length, pond width, pond side slope, pond floor elevation, riser crest elevation, and infiltration rate.

Step 3. Select *Quick Optimization* and Click the *Optimize* button. Quick Optimization will determine a pond configuration, usually in 30 seconds or less, that meets or comes close to meeting the duration design criteria. Quick Optimization is the only option available if sizing an infiltration pond.

Step 4. Return to the *Watershed Layout* tab and adjust the amount of impervious area in the subbasin that contains the detention pond to reflect the pond surface area estimated by the quick optimization routine. Move to the *Compute Runoff* tab and recompute runoff.

Step 5. Return to the *Optimization Tab* of the *Hydraulic Structures* Input Screen and confirm the desired geometry is entered for the pond length, pond width, pond side slope, pond floor elevation, and riser crest elevation.

Step 6. Select either *Quick Optimization* or *Full Optimization* and Click the *Optimize* button. Quick Optimization will determine a pond configuration, usually in 30 seconds or less, that meets or comes close to meeting the duration design criteria. Full Optimization does an exhaustive analysis of pond sizes and outlet works configurations and seeks to find a solution that minimizes the pond volume needed to meet the flow duration standard.

The full optimization option takes from four to ten minutes to complete on an 800MHz processor and will converge to a solution for most project sites. In some situations, usually when precipitation with outliers or precipitation data of poor quality is used, the pond design may not meet all of the design criteria. In these cases, the pond design determined by the program is returned to the Hydraulic Structures and Pond/Vault Geometry tabs for manual refinement. Modifications can be made to the design by the user and flows routed through the pond using manual mode. Guidelines for adjusting the pond size and outlet works are discussed in Section 14.

## 9 Channel Routing

Channel routing is performed using a Modified Puls routing routine developed by the US Army Corps of Engineers for the HEC-1<sup>25</sup> flood hydrograph package. The user inputs the left overbank, main channel and right overbank channel cross sectional geometry, roughness, slope, and channel length (Figure 9.1). The program develops an elevation-volume-discharge rating table assuming normal depth at each discharge level and computes discharge according to the Manning Equation<sup>4</sup>. This rating table is then utilized by the Modified Puls routing routine to route flows from the upstream to the downstream node. To simulate infiltration through the channel bottom, define the link as an infiltration trench and set the depth of gravel to zero (See Section 10).

**Channel Routing Definition (Post): Example Channel**

**Post-Developed Network: New**

Elevation (ft) vs Distance (ft) graph showing the channel profile. Key points: (0, 113.00), (3.0, 107.00), (13.0, 106.00), (16.0, 100.00), (26.0, 100.00), (29.0, 106.00), (39.0, 107.00), (42.0, 113.00).

Channel Name:

☒ View Graph of Input Data  
☐ View Data Input Definitions

LEFT OVERBANK	MAIN CHANNEL	RIGHT OVERBANK
Upper Sideslope (Z)	Lower Sideslope Left (Z)	Upper Sideslope (Z)
<input type="text" value="0.50"/>	<input type="text" value="0.50"/>	<input type="text" value="0.50"/>
Upper Width, W (ft)	Lower Width Left W (ft)	Upper Width W (ft)
<input type="text" value="3.00"/>	<input type="text" value="3.00"/>	<input type="text" value="3.00"/>
Middle Sideslope (Z)	Lower Sideslope Right (Z)	Middle Sideslope (Z)
<input type="text" value="10.00"/>	<input type="text" value="0.50"/>	<input type="text" value="10.00"/>
Middle Width, W (ft)	Lower Width Right W (ft)	Middle Width W (ft)
<input type="text" value="10.00"/>	<input type="text" value="3.00"/>	<input type="text" value="10.00"/>
Mannings n Roughness	Mannings n Roughness	Mannings n Roughness
<input type="text" value="0.040"/>	<input type="text" value="0.024"/>	<input type="text" value="0.040"/>
	Base Width W (ft)	
	<input type="text" value="10.00"/>	
	Base Elevation (ft)	
	<input type="text" value="100.00"/>	
	Channel Bed Slope (ft/ft)	
	<input type="text" value="0.0200"/>	
	Channel Length (ft)	
	<input type="text" value="1000.0"/>	

Ok Cancel

Figure 9.1 – User Input for defining Open Channel Routing



## 10 Infiltration Trench

MGSFlood simulates infiltration using an approach developed by Massmann<sup>30</sup> based on field observations of infiltration ponds in western Washington. This approach accounts for the side slope geometry of the structure, the aspect (length to width ratio), the proximity to the regional groundwater table, and the potential for soil clogging and fouling. Inputs include; Soil Hydraulic Conductivity (inches/hour), Depth to the Regional Water Table (ft), whether bio-fouling potential is low, and whether average or better maintenance is performed. Infiltrated moisture is lost from the system and does not contribute to the discharge rate at the downstream node.

Soil Hydraulic Conductivity (in/hr) – Is the saturated hydraulic conductivity of the soil beneath the infiltration trench in inches per hour according to Darcy's Equation. It can be estimated using regression equations that use grain size distribution as input (Massmann<sup>30</sup>) or from literature (e.g. Freeze and Cherry<sup>31</sup>, Fetter<sup>32</sup>).

Depth to Regional Groundwater Table (ft) – Represents the depth from the bottom of the trench to the regional groundwater table or the first low-permeability layer. For shallow groundwater sites, groundwater mounding reduces the hydraulic gradient and the infiltration rate is significantly less than the saturated hydraulic conductivity. For deep groundwater sites where the effects of mounding will be small, the gradient will not typically be reduced by infiltration from the facility. Increasing the depth to groundwater greater than 100 feet ceases to have an influence on pond infiltration according to this approach.

Bio-fouling Potential – Bio-fouling occurs from organic material blanketing the soil surface and reducing the infiltration rate. Bio-fouling is more likely to occur if the trench is located beneath trees and other vegetation or in shaded locations.

Maintenance – Siltation is more likely to occur if there is not sufficient pre-treatment of the storm water or in locations where the drainage basin is prone to erosion because of recent land disturbances or steep slopes. The user should consider the potential for siltation and the level of maintenance when determining the effects of maintenance on pond infiltration performance.

The program routes flow for two types of infiltration trenches as shown in Figures 10.1 and 10.2; a trench located on the embankment side slope, or an infiltration trench located at the base of the embankment.

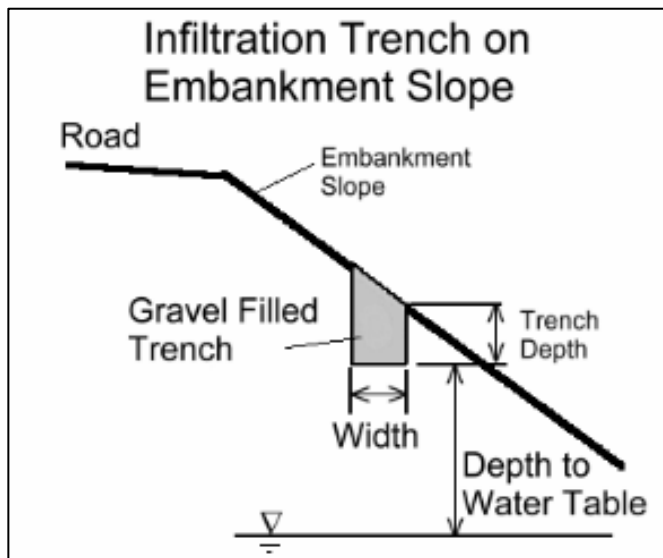


Figure 10.1 – Infiltration Trench Located on Embankment Slope Option

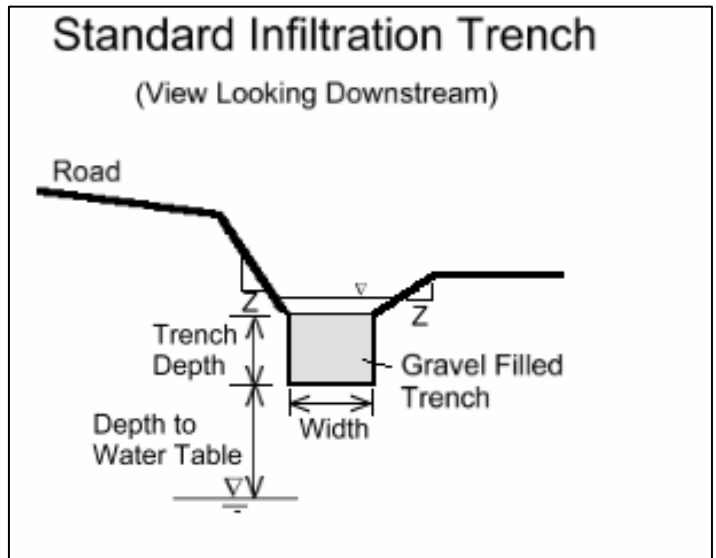


Figure 10.2 – Standard Infiltration Trench Option

### 10.1 Infiltration Trench Located on Embankment Slope

A trench is constructed along the roadway embankment and filled with gravel (Figure 10.1). Runoff from the roadway is directed to the gravel trench where it percolates through the gravel and infiltrates through the trench bottom. When the runoff rate exceeds the infiltration capacity, the gravel saturates from the bottom up with the voids in the gravel providing runoff storage, similar to a detention pond. If the storm is sufficiently large, the saturation will reach the ground surface and runoff from the road will pass over the gravel surface and continue down the embankment. Runoff not infiltrated in the trench is passed to the downstream node without routing.

It should be noted that the saturated hydraulic conductivity of the embankment fill will likely be different from the native material beneath the fill. The hydraulic conductivity estimates of the different layers can be combined using the harmonic mean (Massmann<sup>30</sup>):

$$K_{Equiv} = \frac{d}{\sum \frac{d_i}{K_i}} \quad 10.1$$

Where:  $K_{Equiv}$  is the equivalent hydraulic conductivity,  
 $d$  is the depth of the soil column above the regional groundwater table or limiting permeability layer,  
 $d_i$  is the thickness of layer  $i$ ,  
 $K_i$  is the hydraulic conductivity of layer  $i$

Note that the saturated hydraulic conductivity of the gravel in the trench is not included in Equation 10.1.

For sites with very deep groundwater tables (>100 feet), it is recommended that the total depth of the soil column in Equation 10.1 be limited to 20 times the trench depth.

## **10.2 Standard Infiltration Trench**

The standard infiltration trench would be constructed at the base of the roadway embankment and would receive runoff from the adjacent roadway or from an upstream ditch. Runoff from the roadway is directed to the gravel trench where it percolates through the gravel and infiltrates through the trench bottom. When the runoff rate exceeds the infiltration capacity of the soil, the gravel saturates from the bottom up with the voids in the gravel providing runoff storage, similar to a detention pond. If the storm is sufficiently large, the saturation will reach the ground surface and runoff will occur down the ditch along the gravel surface. The program routes flow along the gravel surface to the downstream node according to the Manning Equation<sup>4</sup>.

The infiltration trench routine may also be used to simulate a natural stream channel with infiltration through the channel bottom. The geometry of the channel is defined as a trapezoidal section and depth of gravel is input as zero.

## **10.3 Automatic Infiltration Trench Sizing Routine**

The automatic pond sizing optimization routine in the MGSFlood will automatically determine the size of infiltration trench required to meet the goals of the Ecology flow duration standard. The optimization routine uses general input about the infiltration trench and solves for the trench bottom width. The input supplied by the user includes:

- ❖ The type of infiltration trench to be sized (Embankment Slope or Standard),
- ❖ The trench bottom elevation at the downstream end,
- ❖ Trench depth,
- ❖ Trench length,
- ❖ Rock fill porosity,
- ❖ Depth to water table,
- ❖ Saturated hydraulic conductivity of soil beneath trench.

The optimization routine uses the information listed above to establish the geometric relationships for the trench configuration. The program establishes a parameter space of possible solutions by varying the bottom width. The program then routes the developed runoff timeseries through the trench and seeks to find a solution that provides the minimum bottom width to meet the duration design standard.

Flow duration curves computed for infiltration trenches typically plot along the horizontal axis and then bend sharply upward. This indicates that all runoff is infiltrated to a point and then the infiltration capacity of the ditch is exceeded and results in surface flow to the downstream node (Figure 10.3)

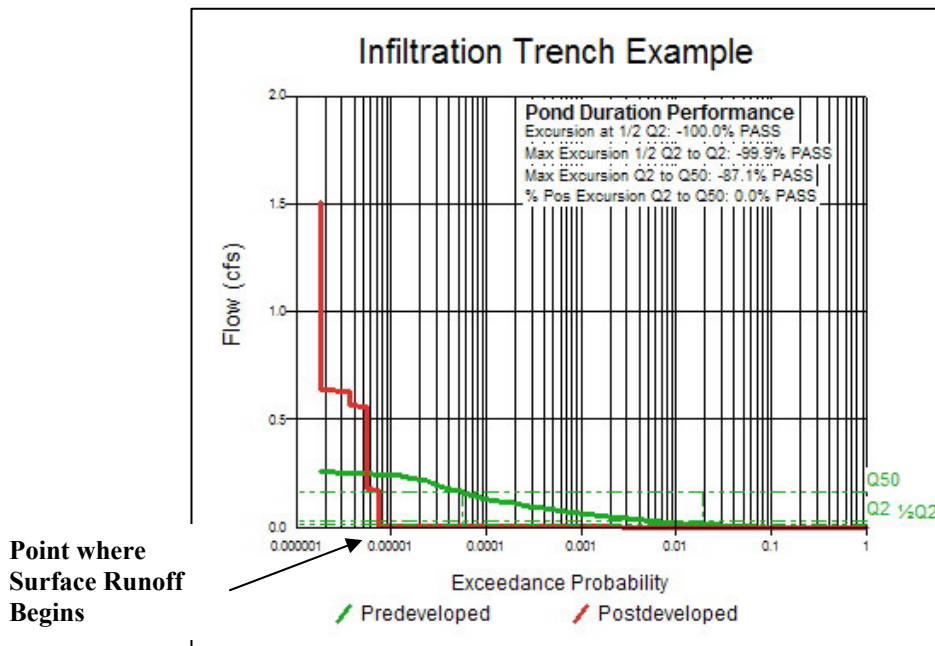


Figure 10.3 – Example Infiltration Trench Flow Duration Curve Performance

## 11 User Defined Rating Table

Structure hydraulics are specified using a stage-surface area-volume-discharge rating table. The pond storage (acre-feet), surface area (acres), discharge (cfs), and infiltration (inches per hour) are computed by the user and entered in the table. Information may be copied from an external spreadsheet program and pasted into the input table using the Windows Clipboard utility.

## 12 Runoff/Network Routing Computation

### 12.1 Overview

After inputting land use, connecting subbasins to nodes, and defining links connecting nodes, runoff and routing computations are performed from the *Runoff/Optimize* tab. MGSFlood computes runoff using the impervious (IMPLND) and pervious (PERLND) land segment subroutines from the HSPF model. Hourly precipitation and evaporation are read from the MGSRegion.mdb file, runoff is computed for predevelopment and postdevelopment conditions, and saved to a FORTRAN, binary, direct access file called TSSRunoff.da. Routing through the predeveloped and postdeveloped networks is then performed with output saved in a FORTRAN direct access file called TSSRoute.da. Statistics are then performed automatically and the results are plotted on the *Graphics* tab.

Runoff computations are performed on a *water year* basis, that is, they begin on October 1 and end on September 30. This is done because the soils are typically driest at the beginning of fall and a single set of antecedent conditions can be used for all regions of western Washington upon startup for the first year of the simulation. The user can define a time period shorter than the full record for the runoff computations, although the full period of record should be used in facility design to provide the most accurate design. The same FORTRAN direct access files are overwritten for each project analyzed by the flood model, i.e. the computed runoff timeseries are not saved for each project.

The program will automatically determine the size of pond or infiltration trench checked on the *Postdeveloped Condition Network* Tab if the *Optimize Structure Indicated on Network tab* option is checked. Only one structure may be optimized per simulation run. To optimize multiple structures, start with the furthest upstream structure and optimize each structure working downstream.

Statistics may be computed for the compliance nodes only (low level output) or all nodes in the project (higher output level). Computed statistics are available for graphing and are saved in the project report.

### 12.2 Governing Equations for Routing

Network routing is performed using a Modified Puls routing routine developed by the US Army Corps of Engineers for the HEC-1<sup>25</sup> flood hydrograph package. A storage indication function (Equation 12.1) is computed from storage and outflow data developed by the program for each structure in the network.

$$STRI(I) = C * \frac{STOR(I)}{\Delta t} + \frac{OUTFL(I)}{2} \quad (12.1)$$

Where: *STRI* is the storage indication in cfs, *STOR* is the storage for a given outflow in acre-ft, *OUTFL* is the outflow in cfs, *C* is the conversion factor from

acre-ft/hour to cfs,  $\Delta t$  is the timestep in hours, and  $I$  is a subscript indicating corresponding values of storage and outflow.

Storage indication at the end of each time interval is given by:

$$STR(2) = STRI(1) + QIN - Q(1) \quad (12.2)$$

Where:  $QIN$  is the average inflow in cfs, and  $Q$  is the outflow in cfs, and subscripts 1 and 2 indicate beginning and end of the current timestep.

The outflow at the end of the time interval is interpolated from a table of storage indication versus outflow. Storage is then computed from:

$$STR = \left( STRI - \frac{Q}{2} \right) * \frac{\Delta t}{C}$$

## 13 Flood Frequency and Duration Statistics

MGSFlood contains routines for computing flood-frequency and flow duration statistics on streamflow and water surface elevation timeseries computed by the program. The following sections describe the flow duration and flow frequency statistics, and the flow duration pond design criteria as required by the Washington State Department of Ecology<sup>9</sup>.

### 13.1 Flow Duration Statistics

Flow duration statistics provide a convenient tool for characterizing streamflow computed with a continuous hydrologic model. Duration statistics are computed by tracking the fraction of time that a specified flow rate is equaled or exceeded. The program does this by dividing the range of flows simulated into discrete increments and then tracks the fraction of time that each flow is equaled or exceeded. For example, Figure 13.1a shows a one-year flow timeseries computed at hourly timesteps from a ten acre forested site and Figure 13.1b shows the flow duration curve computed from this timeseries.

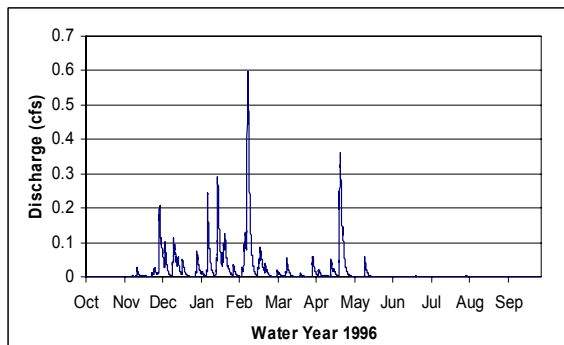


Figure 13.1a – Runoff from 10-Ac Forested Site

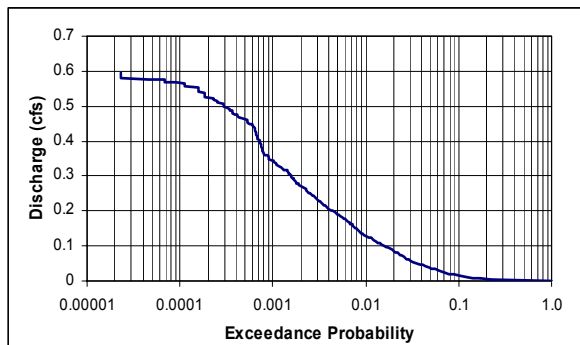


Figure 13.1b – Flow Duration Curve Computed Using Timeseries in Figure 13.1a

The fraction of time that a particular flow is equaled or exceeded is called *exceedance probability*. It should be noted that exceedance probability for duration statistics is different from the *annual exceedance probability* associated with flood frequency statistics and there is no practical way of converting/relating annual exceedance probability statistics to flow duration statistics.

## 13.2 Flood/Water Surface Elevation Frequency Statistics

Flood-frequency analysis seeks to determine the flood flow or water surface elevation with a probability ( $p$ ) of being equaled or exceeded in any given year. Return period ( $Tr$ ) or recurrence interval is often used in lieu of probability to describe the frequency of exceedance of a flood of a given magnitude. Return period and annual exceedance probability are reciprocals (Equation 13.1) and the two are used interchangeably in this section. Flood-frequency analysis is most commonly conducted for flood peak discharge and peak water surface elevation but can also be computed for maximum or minimum values for various durations. Flood-frequency analysis as used here refers to analysis of flood peak discharge or peak water surface elevation.

$$Tr = \frac{1}{p} \quad 13.1$$

Where:

$Tr$  is the average recurrence interval in years, and  
 $p$  is the annual exceedance probability.

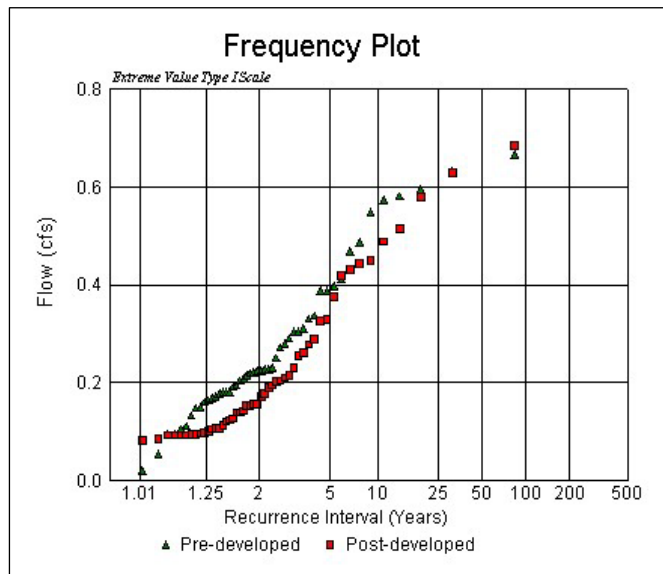
For flows that have not been routed through a stormwater pond, MGSFlood fits the Generalized Extreme Value<sup>14</sup> probability distribution to the annual maxima peak flows using L-Moment<sup>14</sup> statistics. Flow magnitudes for recurrence intervals ranging from 6-months to 500-years are computed with this distribution.

The exceedance probability for streamflow that has been routed through the stormwater pond and water surface elevations are estimated using the Gringorten<sup>12</sup> plotting position formula (Equation 13.2), which is a non-parametric approach. An example probability plot comparing forested land use with the pond outflow is shown in Figure 13.2 and a pond water surface elevation frequency plot is shown in Figure 13.3.

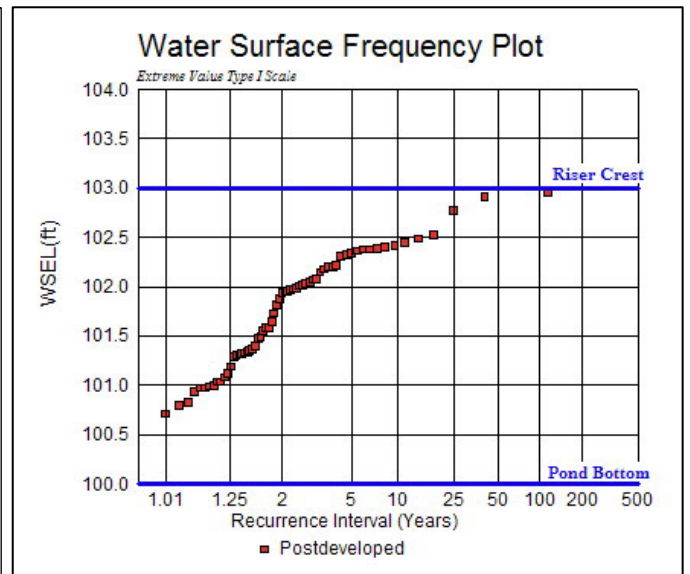
$$Tr = \frac{N + 0.12}{i - 0.44} \quad 13.2$$

Where:  $Tr$  is the recurrence interval of the peak flow or peak elevation in years,  
 $i$  is the rank of the annual maxima peak flow, ranked from highest to lowest, and  $N$  is the total number of years simulated.

A probability distribution, such as the Generalized Extreme Value or Log-Pearson III<sup>15</sup>, is not used for estimating the frequency characteristics because these and other three-parameter distributions typically do a very poor job of fitting annual maxima flows regulated by stormwater ponds and can produce grossly inaccurate estimates of the flow for rare recurrence intervals.



**Figure 13.2 – Example Probability Plot Comparing Pond Outflow (Postdeveloped) with Predeveloped**



**Figure 13.3 – Example Pond Water Surface Elevation Probability Plot**

## 14 Pond Design to Flow Duration Standard

In the past, stormwater pond design criteria have focused on flood control by regulating peak flow rates. Even if the design goal for controlling peak discharge is successful, the aggregate duration that flows occupy the stream channels is greater than under predeveloped conditions because the overall runoff volume is greater under postdevelopment conditions. This increased runoff volume results in increased erosive work being done on the receiving channels, and results in streams that are incised and devoid of the characteristics needed to support fish habitat.

The *flow duration standard* seeks to maintain predevelopment levels of the magnitude and duration of streamflow for those streamflows that exceed the threshold for bedload movement. The threshold for bedload movement is assumed to be 50-percent of the 2-year flow computed for predevelopment conditions<sup>16,2</sup>. The intent of this standard is to prevent increases in the rate of stream channel erosion over that which occurs under predeveloped conditions.

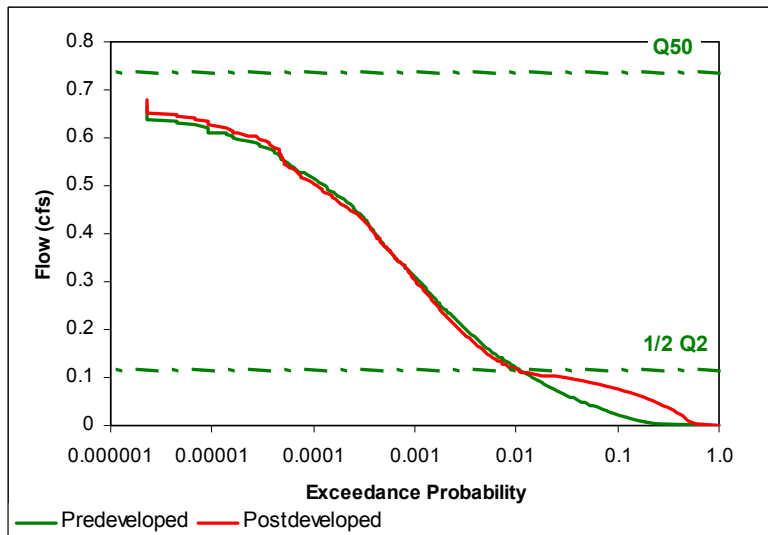
### 14.1 Flow Duration Standard

The following is the flow duration standard required by the Department of Ecology *Stormwater Management Manual for Western Washington*<sup>9</sup>:

*Stormwater discharges shall match developed discharge duration to predeveloped durations for the range of predeveloped discharge rates from 50-percent of the 2-year peak flow up to the full 50-year peak flow.*

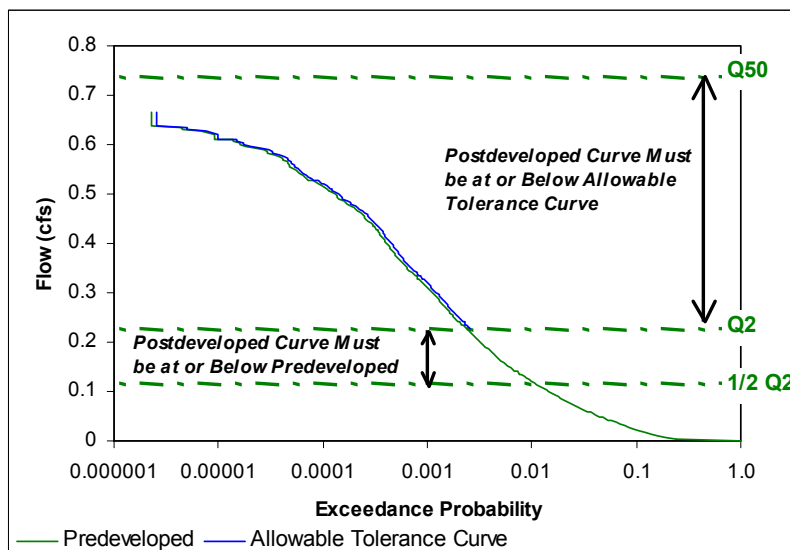
*The pre-developed condition to be matched shall be a forested land cover unless reasonable, historic information is provided that indicates the site was prairie prior to settlement (modeled as pasture). This standard requirement is waived for sites that will reliably infiltrate all the runoff from impervious surfaces and converted pervious surfaces.*

The flow duration standard can be viewed graphically as shown in Figure 14.1. The flow duration curve for the site under predeveloped conditions (forested land cover in this example) is computed and is the target to which the postdeveloped flow duration curve is compared. The flow duration curve for the pond discharge must match the predeveloped curve between  $\frac{1}{2}$  of the predeveloped 2-year ( $\frac{1}{2}$  Q2) and the predeveloped 50-year (Q50). The postdeveloped curve must match the predeveloped within the tolerance levels specified in Table 14.1 and shown graphically in Figure 14.2.



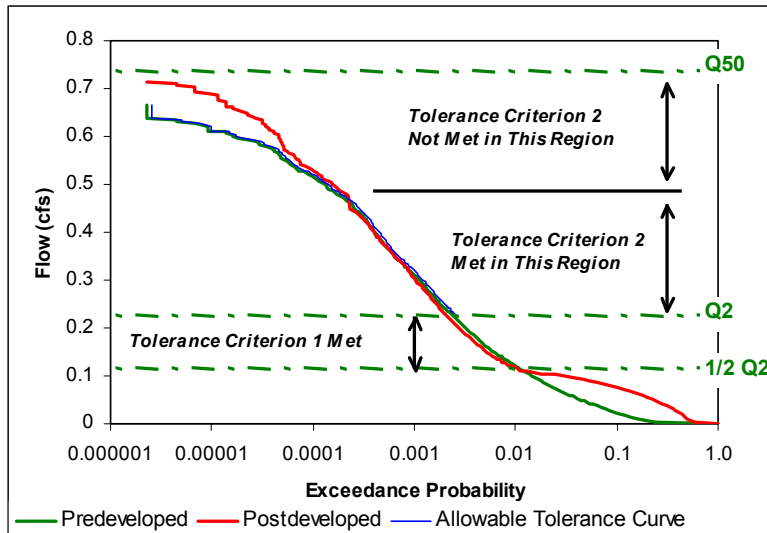
**Figure 14.1 – Comparison of Predeveloped and Postdeveloped Flow Duration Curves**

<b>Table 14.1 – Tolerance Criteria for Matching Postdevelopment Flow Duration Curves to Predevelopment Levels</b>	
1.	The exceedance probability of postdeveloped flow duration values must not exceed the predeveloped values between $\frac{1}{2}$ of the 2-year and the 2-year discharge.
2.	The exceedance probability of postdevelopment flow duration values must not exceed the predeveloped exceedance probability by more than 10% between the 2-year and 50-year discharge.
3.	No more than 50-percent of the postdeveloped flow duration values can be greater than the predeveloped values between $\frac{1}{2}$ Q2 and Q50.



**Figure 14.2 – Criteria for Matching Postdevelopment (Pond Outflow) Duration Curve to Predevelopment Flow Duration Curve**

In the example shown in Figure 14.3, Tolerance Criterion 1 is met because the postdeveloped flow duration curve is at or below the predeveloped between  $\frac{1}{2}$  of the 2-year and the 2-year. Tolerance Criterion 2 is not met, because postdeveloped flow duration curve exceeds the tolerance curve above 0.45 cfs. Tolerance Criterion 3 is met because more than 50-percent of the postdeveloped duration values are at or below the predeveloped curve. Because not all three of the criteria are met, the pond does not meet the flow duration standard and modifications would be needed to the pond size and/or outlet works to meet the standard.



**Figure 14.3 – Predevelopment and Postdevelopment (Pond Outflow) Flow Duration Curves and Flow Duration Standard Performance Criteria (Pond Fails Criterion 2, and Does not Meet Flow Duration Standard)**

## 14.2 Pond/Infiltration Trench Design Procedure

The procedure for designing a stormwater pond or infiltration trench to meet the flow duration standard discussed in Section 13.1 is described in the following sections.

Step 1. Input land use on the *Land Use* tab. Connect subbasins to Nodes.

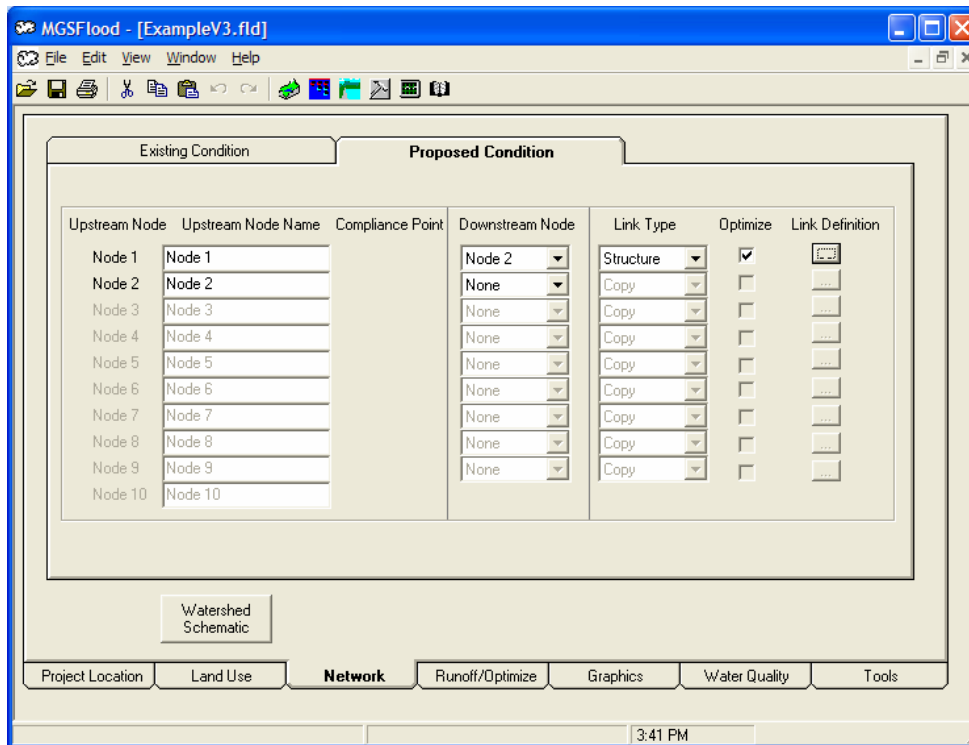
Step 2. Connect upstream nodes to downstream nodes on the *Network* tab. Define the type of link present (copy, structure, infiltration trench, etc) for all links in the predeveloped and postdeveloped network.

Step 3. On the post development network tab, check which link is to be optimized. Only structures and infiltration trenches may be optimized. Click the Link Definition button to input optimization information for the link to be optimized (Figure 14.4).

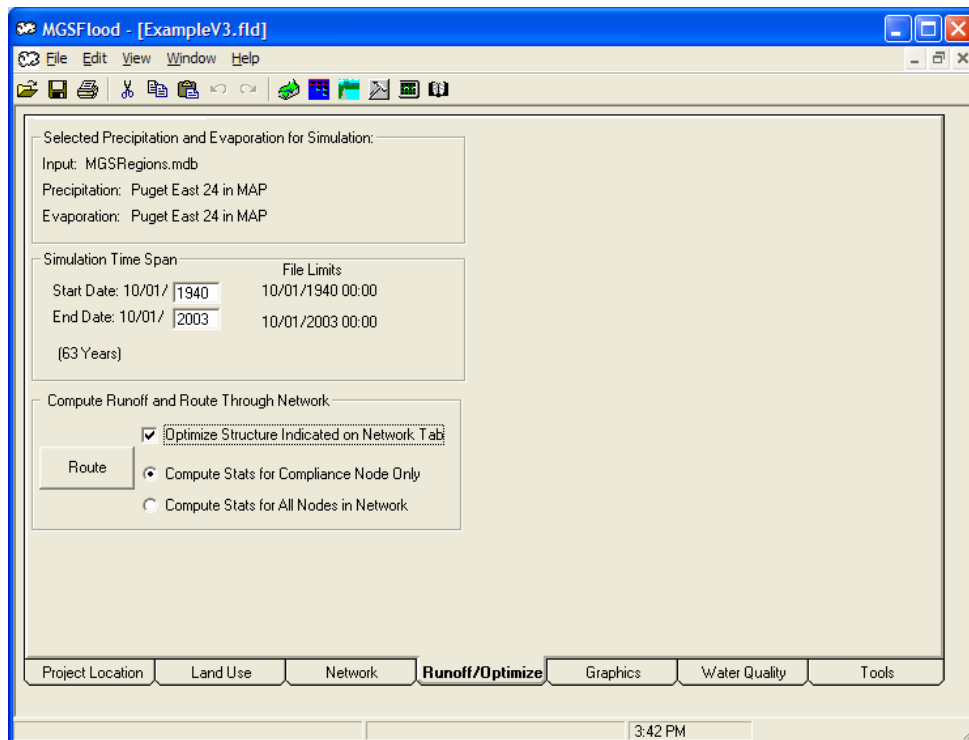
Step 4. Define the predeveloped compliance node. The program will automatically size the pond or infiltration trench such that the flow duration standard is met at the node downstream of the optimized link.

Step 7. On the *Runoff/Optimize* tab, check the *Optimize Structure on the Network Tab* then click the *Route Flows* button (Figure 14.5). The program will compute runoff, route flows through the network for pre and post developed conditions, then iterate and determine the size of structure to meet the flow duration standard. When the iterations are complete, the program will plot duration statistics for the pond outflow for comparison with the compliance node duration curve. Compliance criteria will also be displayed on the graph.

Step 8. If any of the criteria are not met, then the pond configuration must be modified and routing repeated. Subsequent routing to refine the pond design should be performed with the *Optimize Structure on the Network Tab* unchecked. Guidelines for adjusting the pond size and outlet works are discussed in the following section.



**Figure 14.4 – Postdeveloped Network with Pond Optimization to Occur Upstream of Node 2**



**Figure 14.5 – Runoff/Optimization Screen, Clicking the Route Button with the Optimize Box Checked with Route all Flows and Optimize the Structure Indicated on the Network Tab**

### 14.3 Guidelines for Adjusting Pond Performance

General guidance for adjusting the geometry and outlet works of stormwater ponds to meet the duration standard were developed by King County<sup>16</sup>, are summarized in Figure 14.6, and described below. Refinements should be made in small increments with one refinement at a time.

1. *Bottom Orifice Size* – Adjust the bottom orifice to control the lowest arc of the postdeveloped flow duration curve. Increase the orifice size to raise the arc, decrease it to lower the arc.
2. *Height of Second Orifice* – The invert elevation of the second orifice affects the point on the flow duration curve where the transition (break in slope) occurs from the curve produced by the low-level orifice. Lower the invert elevation of the second orifice to move the transition point to the right on the lower arc. Raise the height of the second orifice to move the transition point to the left on the lower arc.
3. *Second Orifice Size* – Adjust to control the arc of the curve for postdeveloped conditions. Increase the size to raise the arc, decrease it to lower the arc.
4. *Pond Volume* – Adjust the pond volume to control the upper end of the duration curve. Increase the volume to prevent overflow, decrease the volume if the duration curve is substantially below the overflow level.

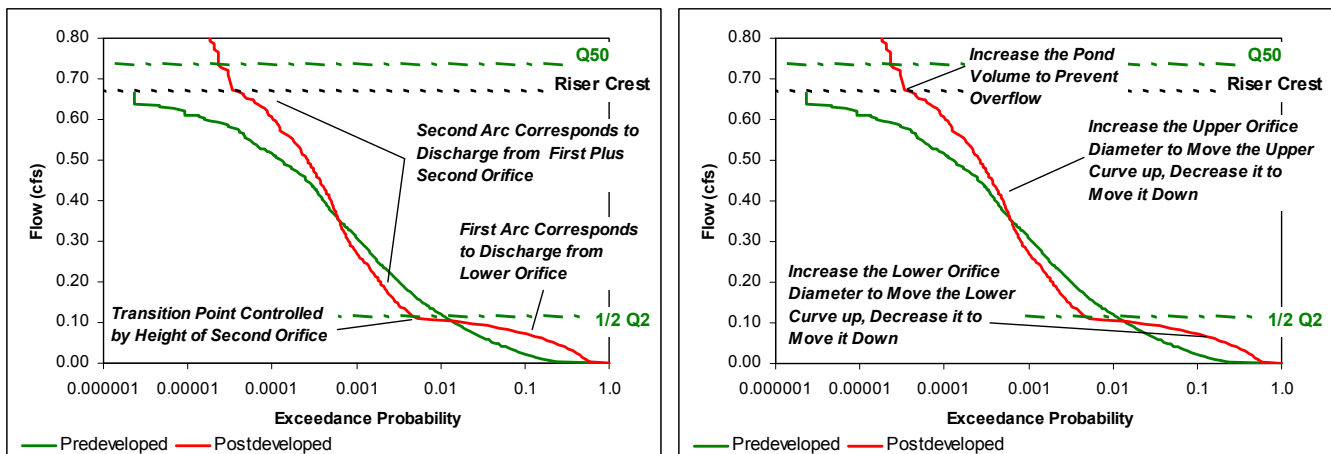


Figure 14.6 – General Guidance for Adjusting Pond Performance

- Analyze the duration curve from bottom to top, and adjust orifices from bottom to top.
- The bottom arc corresponds with the discharge from the bottom orifice. Reducing the bottom orifice discharge lowers and shortens the bottom arc while increasing the bottom orifice raises and lengthens the bottom arc.
- Inflection points in the outflow duration curve occur when additional structures (orifices, notches, overflows) become active.

- Lowering the upper orifice moves the transition right on the lower arc and raising the upper orifice moves the breakpoint left of the lower arc
- The upper arc represents the combined discharge of both orifices. Adjustments are made to the second orifice as described above for the bottom orifice.
- Increasing the facility volume moves the entire curve down and to the left. This is done to control riser overflow conditions. Decreasing facility volume moves the entire curve up and to the right. This is done to ensure that the outflow duration curve extends up to riser overflow.

## 15 Project Documentation/Reporting


The project reporting utility creates a report that documents all model inputs, stormwater pond design information, and frequency and duration statistics. The report is created and viewed on screen by selecting *View Report* from the *File* menu or from the *View Report* icon () on the tool bar. Note that the View Report utility only becomes active after saving the project file for the first time. The report can be printed by selecting *Print Report* from the File menu. When the project report is printed, any frequency and duration graph images stored in the project directory are also printed. These images represent the latest graphs plotted from the *Graphs* tab. Each time the report is viewed or printed, a copy of the report is stored in a file with the name *<ProjectName.rtf>* in the project data directory. This file is a Windows Rich Text Format (RTF) and can be edited with Microsoft Word or Word Pad. A partial listing of a project report is shown below.

Figure 15.1 – Project Report Output (Partial Listing)

---

**MGS FLOOD  
PROJECT REPORT**

Program Version: 3.0                      Run Date: 06/26/2005 1:21 PM

---

**Input File Name:**            **ExampleV3.fld**  
**Project Name:**  
**Analysis Title:**  
**Comments:**

\*\*\*\*\* **Precipitation Input**\*\*\*\*\*

Extended Precipitation Timeseries Selected  
Climatic Region Number: 9

**\*\*\* WARNING \*\*\* FULL PERIOD OF RECORD NOT USED FOR ROUTING**  
**Available Precipitation and Evaporation Data:**    10/01/1940-10/01/2003  
**Data Used in Analysis:**  
Precipitation Station : 960024 Puget East 24 in MAP                      10/01/1948-10/01/1948  
Evaporation Station : 961024                      Puget East 24 in MAP  
Evaporation Scale Factor : 0.750

HSPF Parameter Region Number: 1  
HSPF Parameter Region Name : USGS Default

\*\*\*\*\* Default HSPF Parameters Used (Not Modified by User) \*\*\*\*\*

**\*\*\*\*\* Watershed Definition \*\*\*\*\***

Number of Subbasins: 1

**\*\*\*\*\* Subbasin Number: 1 \*\*\*\*\***

\*\*\*Tributary to Node: 1

\*\*\*Bypass to Node : None

	-----Area(Acres) -----		
	Predeveloped	-----Developed-----	
		To Node	Bypass Node
<b>Till Forest</b>	0.000	0.000	0.000
Till Pasture	10.000	0.000	0.000
Till Grass	0.000	0.000	0.000
Outwash Forest	0.000	0.000	0.000
Outwash Pasture	0.000	0.000	0.000
Outwash Grass	0.000	0.000	0.000
Wetland	0.000	0.000	0.000
Impervious	0.000	10.000	0.000
<b>Subbasin Total</b>	<b>10.000</b>	<b>10.000</b>	<b>0.000</b>

	-----Lateral Flow Connections-----					
	-----Predeveloped-----			-----Developed-----		
	Surface	Interflow	Grnd Water	Surface	Interflow	Grnd Water
<b>Till Forest</b>	Node	Node		Node	Node	
Till Pasture	Node	Node		Node	Node	
Till Grass	Node	Node		Node	Node	
Outwash Forest	Node	Node		Node	Node	
Outwash Pasture	Node	Node		Node	Node	
Outwash Grass	Node	Node		Node	Node	
Wetland	Node	Node		Node	Node	
Impervious	Node			Node		
Lateral 1						
Lateral 2						
.						
.						
.						

**\*\*\*\* Flow Duration Performance According to Dept. of Ecology Criteria \*\*\*\***

Excursion at Predeveloped ½Q2 (Must be Less Than 0%):	-21.4%	PASS
Maximum Excursion from ½Q2 to Q2 (Must be Less Than 0%):	-20.6%	PASS
Maximum Excursion from Q2 to Q50 (Must be less than 10%):	8.6%	PASS
Percent Excursion from Q2 to Q50 (Must be less than 50%):	3.8%	PASS

---

**\* POND MEETS ALL DURATION DESIGN CRITERIA: PASS**

---

## 16 Exporting Runoff Timeseries

Runoff timeseries computed by the program are stored in binary direct access files. These timeseries can be exported to an ASCII formatted file from the *Tools* tab. The output frequency option defines the number of time intervals to be aggregated before output is written to the file. For example, if the *Daily* option button is selected, then the timeseries will be aggregated and saved to the file once per day. For runoff computed on an hourly time-step, 24 values will be aggregated according to the option selected in the *Display* box. If *Maximum* were selected, then the maximum daily flow would be output, *Minimum* would result in the minimum daily flow, and *Average* would result in the average daily flow.

The output file format consists of the end of period date and time followed by the pre and post developed flows at each node (Figure 16.1). Link inflow, outflow, infiltrated moisture and water surface elevation can also be output for each link in the project (Figure 16.2). Water surface elevation data computed by the high groundwater Perlnd routine may also be exported if present in the project (Figure 16.3).

Date	Node 1(cfs)		Node 2(cfs)	
	Predev	PostDev	Predev	PostDev
10/02/1940 00:00	6.8814E-04	8.0156E-01	6.8814E-04	9.1182E-03
10/03/1940 00:00	1.3172E-04	2.0747E-03	1.3172E-04	9.0165E-03
10/04/1940 00:00	6.5862E-05	2.7013E-04	6.5862E-05	8.5383E-03
10/05/1940 00:00	3.2931E-05	8.7715E-05	3.2931E-05	8.0520E-03
10/06/1940 00:00	1.6466E-05	1.9949E-03	1.6466E-05	7.5884E-03
10/07/1940 00:00	8.2328E-06	0.0000E+00	8.2328E-06	7.1540E-03
10/08/1940 00:00	0.0000E+00	0.0000E+00	0.0000E+00	6.7027E-03
10/09/1940 00:00	0.0000E+00	0.0000E+00	0.0000E+00	6.1584E-03
10/10/1940 00:00	0.0000E+00	0.0000E+00	0.0000E+00	5.6583E-03
10/11/1940 00:00	3.9980E-04	7.6080E-01	3.9980E-04	1.2096E-02

Figure 16.1 – Example Output Produced by Export Utility (Node Output)

----- Postdeveloped Link 1 -----				
Date	Inflow (cfs)	Outflow (cfs)	InfiltQ (cfs)	WSEL (FT)
10/02/1940 00:00	8.0156E-01	9.1182E-03	0.0000E+00	1.0011E+02
10/03/1940 00:00	2.0747E-03	9.0165E-03	0.0000E+00	1.0011E+02
10/04/1940 00:00	2.7013E-04	8.5383E-03	0.0000E+00	1.0010E+02
10/05/1940 00:00	8.7715E-05	8.0520E-03	0.0000E+00	1.0009E+02
10/06/1940 00:00	1.9949E-03	7.5884E-03	0.0000E+00	1.0008E+02
10/07/1940 00:00	0.0000E+00	7.1540E-03	0.0000E+00	1.0007E+02
10/08/1940 00:00	0.0000E+00	6.7027E-03	0.0000E+00	1.0006E+02
10/09/1940 00:00	0.0000E+00	6.1584E-03	0.0000E+00	1.0005E+02
10/10/1940 00:00	0.0000E+00	5.6583E-03	0.0000E+00	1.0004E+02
10/11/1940 00:00	7.6080E-01	1.2096E-02	0.0000E+00	1.0019E+02

Figure 16.2 – Example Output Produced by Export Utility (Link Output)

Predeveloped Perlnd No. 1: Till Grass  
 Postdeveloped Perlnd No. 1: Till Grass  
 Postdeveloped Perlnd No. 2: Lateral 1

Date	Water Surface Elevation (ft)		
	Predev	Postdev	Postdev
	Sub 1, Per 1	Sub 1, Per 1	Sub 1, Per 2
10/02/1939 00:00	1.0000000E+02	1.0000000E+02	1.0000000E+02
10/03/1939 00:00	1.0000000E+02	1.0000000E+02	1.0000000E+02
10/04/1939 00:00	1.0000000E+02	1.0000000E+02	1.0000000E+02
10/05/1939 00:00	1.0000000E+02	1.0000000E+02	1.0000000E+02
10/06/1939 00:00	1.0000000E+02	1.0000000E+02	1.0000000E+02
10/07/1939 00:00	1.0000000E+02	1.0000000E+02	1.0000000E+02
10/08/1939 00:00	1.0000000E+02	1.0000000E+02	1.0000000E+02
10/09/1939 00:00	1.0000000E+02	1.0000000E+02	1.0000000E+02
10/10/1939 00:00	1.0000000E+02	1.0000000E+02	1.0000000E+02
10/11/1939 00:00	1.0000000E+02	1.0000000E+02	1.0000000E+02
10/12/1939 00:00	1.0000000E+02	1.0000000E+02	1.0000000E+02

**Figure 16.3 – Example Output Produced by Export Utility  
 (High Groundwater Perlnd Water Surface Elevation)**

## **17 Water Quality Treatment Design Data**

MGSFlood determines water quality treatment design parameters from the computed runoff timeseries according to methods defined in the 2005 Department of Ecology Stormwater Management Manual for Western Washington<sup>9</sup>. The user should refer to the Ecology Stormwater Manual for specific information regarding water quality treatment requirements and design methods.

Two types of water quality treatment parameters are computed by MGSFlood;

- Water Quality Design Volume, used for sizing wet ponds,
- Water Quality Design Flow Rate, used for sizing flow rate dependent facilities such as biofiltration swales and filter strips.

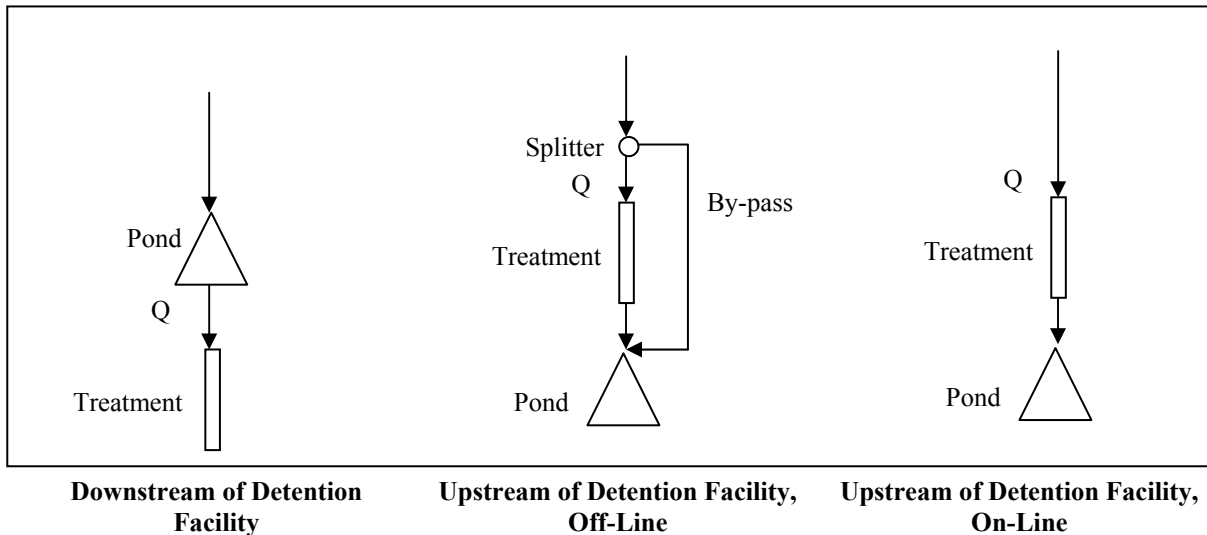
### **17.1 Water Quality Design Volume**

The water quality design volume for sizing wet ponds is computed as the 91% non-exceedance 24-hour runoff volume. The program develops a daily runoff timeseries from the hourly pond inflow timeseries and scans the computed daily timeseries to determine the 24-hour volume that is greater than or equal to 91% of all daily values in the timeseries. According to the Ecology Stormwater Management Manual, this value is then used as the volume for a “Basic Wet Pond” and 1.5 times this value is used for sizing a “Large Wet Pond.” These values are computed automatically at the time runoff is computed for the detention facility inflow and are listed on the Water Quality Data tab and in the project summary report.

## 17.2 Water Quality Design Discharge

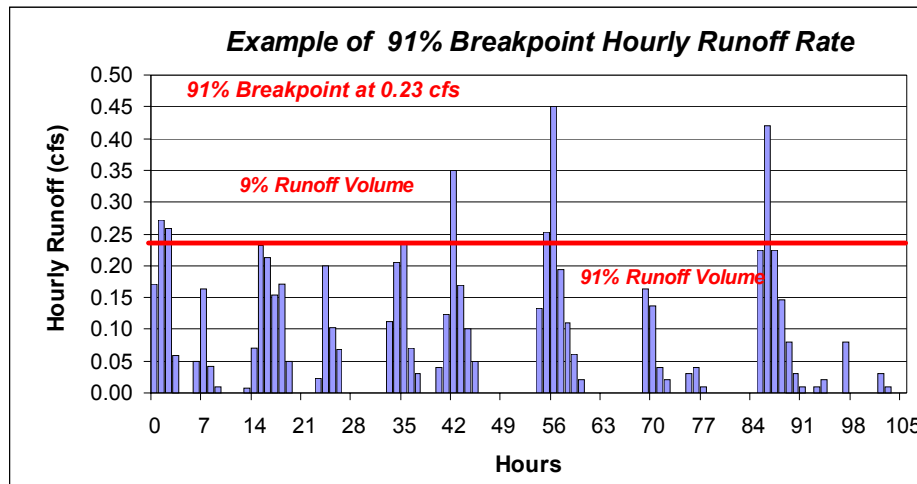
The flow rate used to design flow rate dependent treatment facilities depends on whether or not the treatment is located upstream of a stormwater detention facility and whether it is an *on-line* or *off-line* facility (Figure 17.1).

Figure 17.1



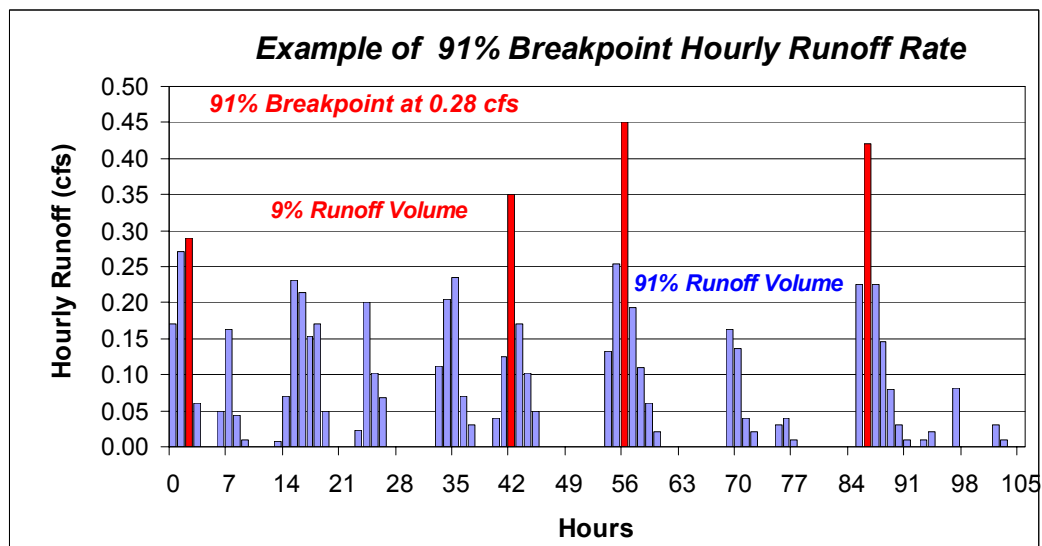
Downstream of Detention Facilities – If the treatment facility is located downstream of a stormwater detention facility, then the full 2-year release rate from the stormwater pond should be used to design the stormwater treatment facility.

Upstream of Detention Facilities, Off-Line – *Off-line* water quality treatment located upstream of the detention facility includes a high-flow by-pass that routes the incremental flow in excess of the water quality design rate around the treatment facility. It is assumed that flows from the bypass enter the system downstream of the treatment facility but upstream of the detention facility. Thus, the bypass does not have to be accounted for in Node Definitions of the Watershed Layout tab. The program determines the hourly water quality treatment design flow rate as the rate corresponding to the runoff volume that is greater than or equal to 91% of the hourly runoff volumes (Figure 17.2). The 15-minute water quality treatment design flow rate is based on an adjustment factor provide by Ecology for estimation of maximum 15-minute flow rates based on hourly timeseries.



**Figure 17.2 – Example showing calculation of Off-Line Water Quality Treatment Discharge**  
**Off-line Hourly Discharge of 0.23 cfs (in this case) is Automatically Adjusted**  
**by the Program to Obtain 15-minute Discharge Rate Used for Design**

Upstream of Detention Facilities, On-Line – On-line water quality treatment does not include a high-flow bypass for flows in excess of the water quality design flow rate and all runoff is routed through the facility. The program determines the hourly water quality treatment design flow rate as the rate corresponding to the runoff volume that is greater than or equal to 91% of the hourly runoff volume entering the treatment facility, however, those flows that exceed the water quality design flow are not included in the calculation (Figure 17.3). Thus, the design flow rate for on-line facilities is higher than for off-line facilities. As discussed above, the 15-minute water quality treatment design flow rate is based on an adjustment factor provide by Ecology.



**Figure 17.3 – Example showing calculation of Off-Line Water Quality Treatment Discharge**  
**Off-line Hourly Discharge of 0.28 cfs (in this case) is Automatically Adjusted**  
**by the Program to Obtain 15-minute Discharge Rate Used for Design**

### 17.3 Water Quality Flow Splitter Design

When an *off-line* treatment approach is used, a flow-splitter is needed for bypassing flows that exceed the design flow rate. MGSFlood computes the geometry of the splitter structure according to guidelines listed in the Ecology Stormwater Management Manual. The splitter structure includes an orifice and an overflow weir (Figure 17.4). The design guidelines are listed below.

- The maximum head on the overflow weir must be minimized for flow in excess of the water quality design flow. Specifically, flow to the water quality facility at the 100-year water surface must not increase the design water quality flow by more than 10-percent.
- The splitter structure requires an orifice plate upstream of the discharge pipe that leads to the water quality treatment facility. The design water surface should be set to provide a minimum headwater/diameter ratio of 2.0.

The splitter design is a trial and error procedure whereby the orifice diameter is selected by the user. The program then computes the height of the baffle wall, the length of the overflow weir, and the ratio of the baffle wall height to orifice diameter. There is not a unique solution and the user should select an orifice size that produces a baffle wall height and overflow length that will conveniently fit in a standard manhole (or other structure) and meets the required headwater/diameter ratio of 2.0.

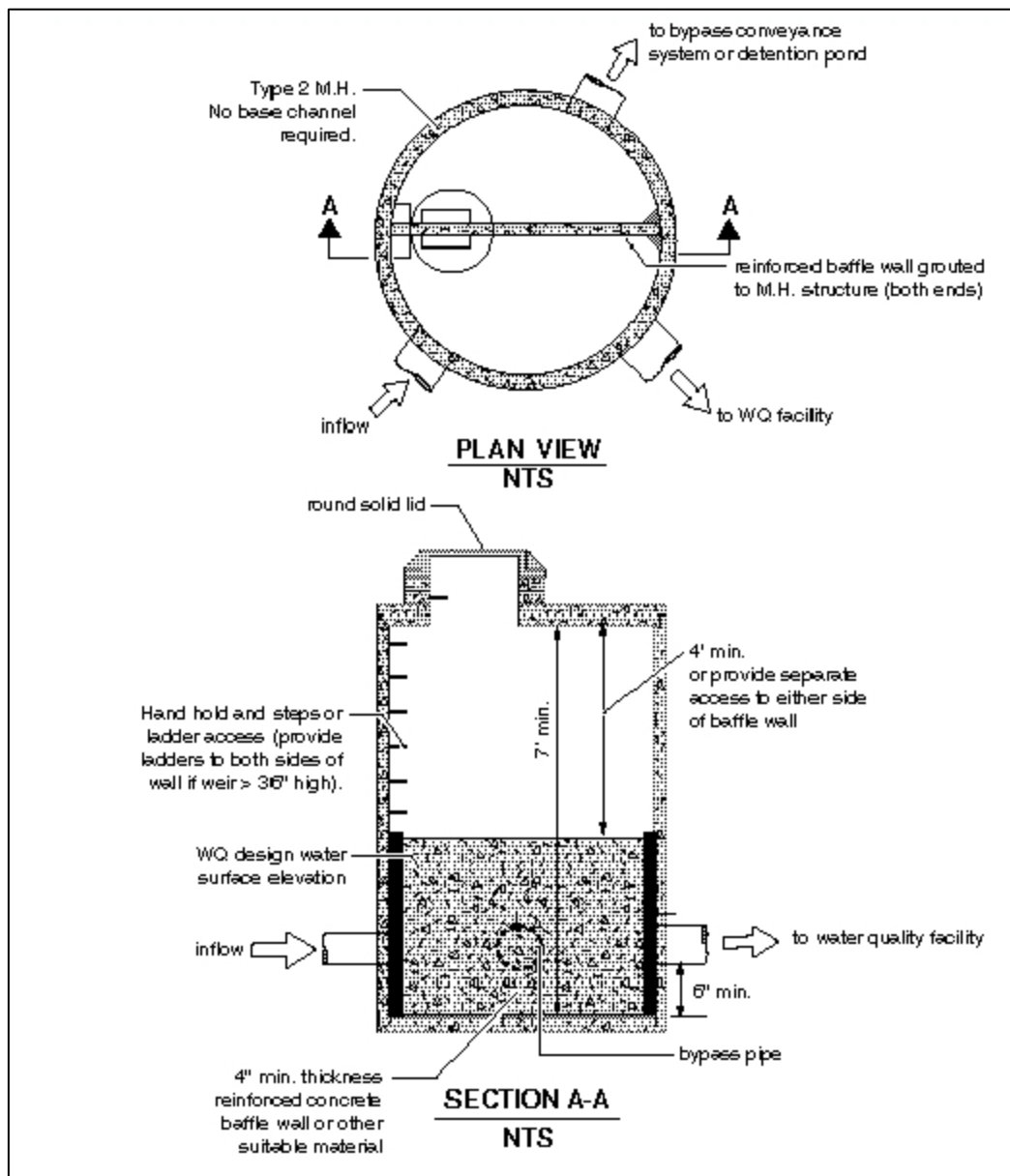


Figure 17.4 – Flow Splitter Geometry (per Ecology Stormwater Management Manual)



## 18 Wetland Water Level Analysis

### 18.1 Introduction

Protection of wetland plant and animal communities depends on controlling the wetland's *hydroperiod*, meaning the pattern of fluctuation of water depth and the frequency and duration of exceeding certain levels, including the length and onset of drying in the summer.

MGSFlood computes hydroperiod statistics according to the guidance developed by the Puget Sound Wetlands and Stormwater Management Research Program<sup>33</sup>. The wetland water level fluctuation guidelines (Guide Sheet 2: Wetland Protection Guidelines) were adopted by Ecology and are listed in Appendix D of the Volume I of Ecology's Stormwater Management Manual<sup>9</sup>. The following sections summarize the water level fluctuation statistics computed by MGSFlood.

### 18.2 Water Level Fluctuation (WLF)

Methods for computing Water Level Fluctuation (WLF) were not defined for continuous flow hydrologic models by the Puget Sound Stormwater Management Research Program. Instead, WLF was defined in terms of data collected using a crest stage gage. A crest stage gage consists of a staff gage for observing the instantaneous water surface elevation in the wetland. The gage also indicates the maximum water surface that occurs between observations.

WLF was defined by the Puget Sound Stormwater Management Research Program in terms of crest stage observations made no more than one month apart as follows:

$$\text{WLF} = \text{Crest stage} - \text{Average base stage} \quad 18.1$$

Where: Crest stage= Maximum stage during interval

Average base stage =  $(\text{Stage}_1 + \text{Stage}_2)/2$

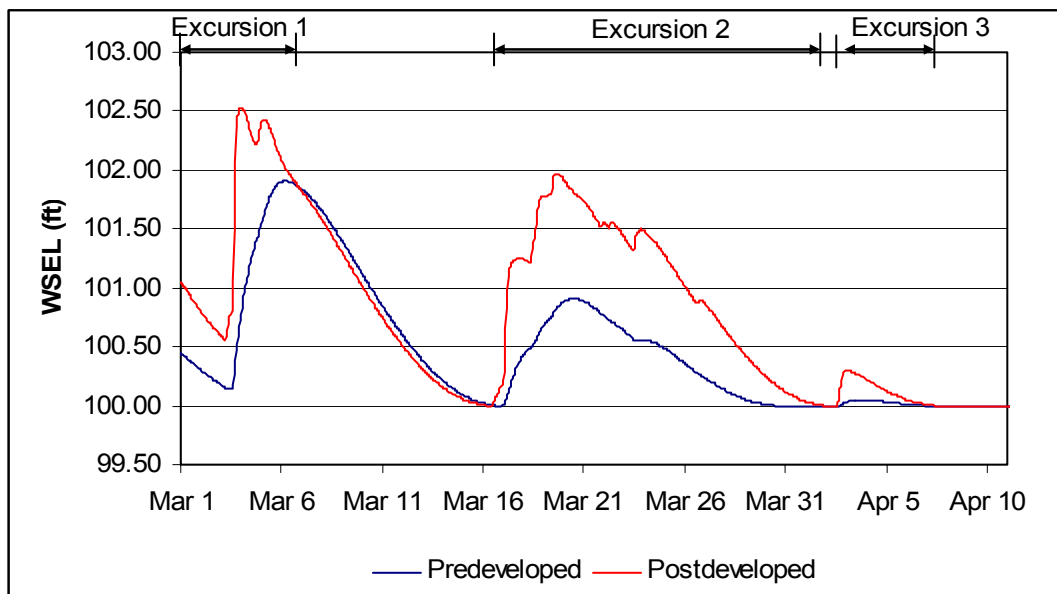
$S_1$ = Instantaneous stage at beginning of interval

$S_2$ = Instantaneous stage at end of interval

This definition was adapted for use with the continuous flow model by using wetland water surface elevation information simulated by the model. Each month was divided into four periods with WLF computed according to Equation 18.1 for the entire simulation period. Average monthly and average annual WLF statistics are then computed and printed in the project report.

### 18.3 Stage Excursions

Stage excursions are defined as the difference between the predeveloped and postdeveloped water surface elevation above a specified threshold. The default threshold is 15 cm (0.5 feet). Thus, each time that the absolute value of the difference between simulated predeveloped and postdevelopment water surface elevation exceeds the threshold, then an excursion begins. When the difference drops below the threshold, then the excursion ends. Figure 18.1 shows a portion of the simulated predeveloped and postdeveloped wetland water surface elevation timeseries for an example wetland. Three excursions are indicated. Each excursion denotes a period here the difference between the predeveloped and postdeveloped timeseries exceeds the 0.5 foot threshold.



**Figure 18.1 – Example Predeveloped and Postdeveloped Wetland Water Surface Elevation with Stage Excursions Noted (Stage Excursion Threshold = 0.5 feet)**

The program computes stage excursions for the entire simulation period and outputs several excursion statistics in the project report. These include:

- ❖ Number of stage excursions per year,
- ❖ The total duration of excursions per year,
- ❖ The average duration of each excursion per year,
- ❖ The maximum excursion for each year,
- ❖ The duration of the longest excursion during the year.

### 18.4 Dry Period Analysis

The program tracks the number of hours per year that the water surface elevation drops below a user specified value. The default value for “dry” conditions is a depth less than 0.01 feet. The statistic is computed for both predeveloped and postdeveloped conditions and reported for each year simulated.

## 18.5 Amphibian Breeding Period Analysis

The program computes hydroperiod limits for a user specified amphibian breeding period (default February 1<sup>st</sup> through May 31). The program reports the duration of stage excursions above or below the predevelopment level in continuous 30-day periods during the breeding months. The default stage excursion threshold is 8 cm (0.25 feet). These statistics allow for the evaluation amphibian criteria which states that the magnitude of stage excursions above or below the predevelopment stage should not exceed 8 cm for more than 24 hours in any 30-day period.

Example wetland hydroperiod statistics computed by the program are shown in Figure 18.2.

**Figure 18.2 – Example Wetland Hydroperiod Analysis Output**

Predeveloped Wetland Location: Link 1: Predeveloped Test Wetland				
Postdeveloped Wetland Location: Link 1: Post Developed Condition Wetland				
*****Mean Water Level Fluctuation Results (ft) *****				
Month	Predeveloped	Postdeveloped		
Oct	0.0033	0.2971		
Nov	0.0162	0.4894		
Dec	0.0943	0.4932		
Jan	0.1663	0.5036		
Feb	0.1294	0.4206		
Mar	0.0797	0.3413		
Apr	0.0253	0.2208		
May	0.0047	0.1517		
Jun	0.0009	0.1458		
Jul	0.0001	0.0698		
Aug	0.0000	0.0802		
Sep	0.0001	0.1808		
Ann	0.0433	0.2829		
*****Stage Excursion Results *****				
Stage Excursions Threshold (ft): 0.500				
Avg Number of Stage Excursions Per Year: 13.824				
WY	No. Excursions	Max (ft)	Max Dur (hrs)	Avg Duration (hrs)
1940	16	2.4519	430.0	111.1
1941	15	1.7901	240.0	70.8
1942	12	2.1804	310.0	89.6
1943	10	2.4195	661.0	162.9
1944	11	1.5237	179.0	50.1
1945	18	2.5822	257.0	66.0
1946	15	1.6662	391.0	95.5
...				
*****No Water (Dry) Excursion Results *****				
Wetland Dry when Stage Drops Below (ft): 0.010				
Dry Excursion Duration (hrs)				
WY	Predeveloped	Postdeveloped		
1940	.0	4568.0		
1941	.0	4096.0		
1942	.0	4727.0		
1943	.0	5244.0		
1944	.0	5692.0		
1945	.0	4792.0		
1946	.0	4583.0		
...				

\*\*\*\*\*Amphibian Season Analysis\*\*\*\*\*

Season Begins : 02/01

Season Ends : 05/31

Amphibian Stage Excursions Threshold (ft): 0.250

WY	Max Excursion (ft)	Max 30-Day Excursion (hrs)
1940	1.173	630.0
1941	0.935	163.0
1942	0.826	289.0
1943	1.938	444.0
1944	0.713	158.0
1945	2.582	454.0
1946	1.666	480.0

...

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# PART II – PROGRAM OPERATION AND DATA INPUT

## 1 Purpose

MGS Flood is a general, continuous, rainfall runoff computer model developed for stormwater facility design in western Washington. Specifically, the program is intended to size stormwater detention ponds to meet the requirements of the 2005 Washington State Department of Ecology Stormwater Management Manual for Western Washington<sup>9</sup>. The program uses the Hydrological Simulation Program-Fortran (HSPF)<sup>26</sup> routine for computing runoff from rainfall.

## 2 Computer Requirements

- Windows 9x/2000/XP with 300 MB uncompressed hard drive space.
- Pentium 4, 1000 MHz or faster processor (desirable).
- The program is designed to be installed and operated from a single computer and not run from a network.

## 3 Detention Pond Sizing Overview

The MGSFlood input screen (Figure 3.1) is organized as a series of tabs that follows the sequence of steps to analyze stormwater runoff. These steps include:

- Entering the project information and determining the precipitation and runoff parameters,
- Entering the subbasin land use information and node connections,
- Entering Network Connections, Defining Links (ponds, trenches, channels),
- Routing Flows,
- Plotting pond performance graphs.

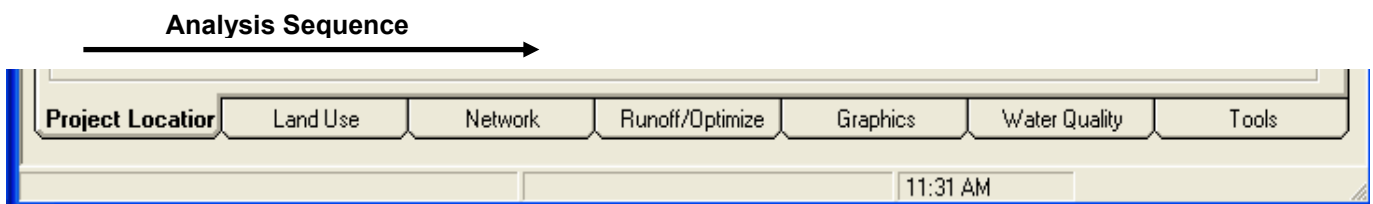
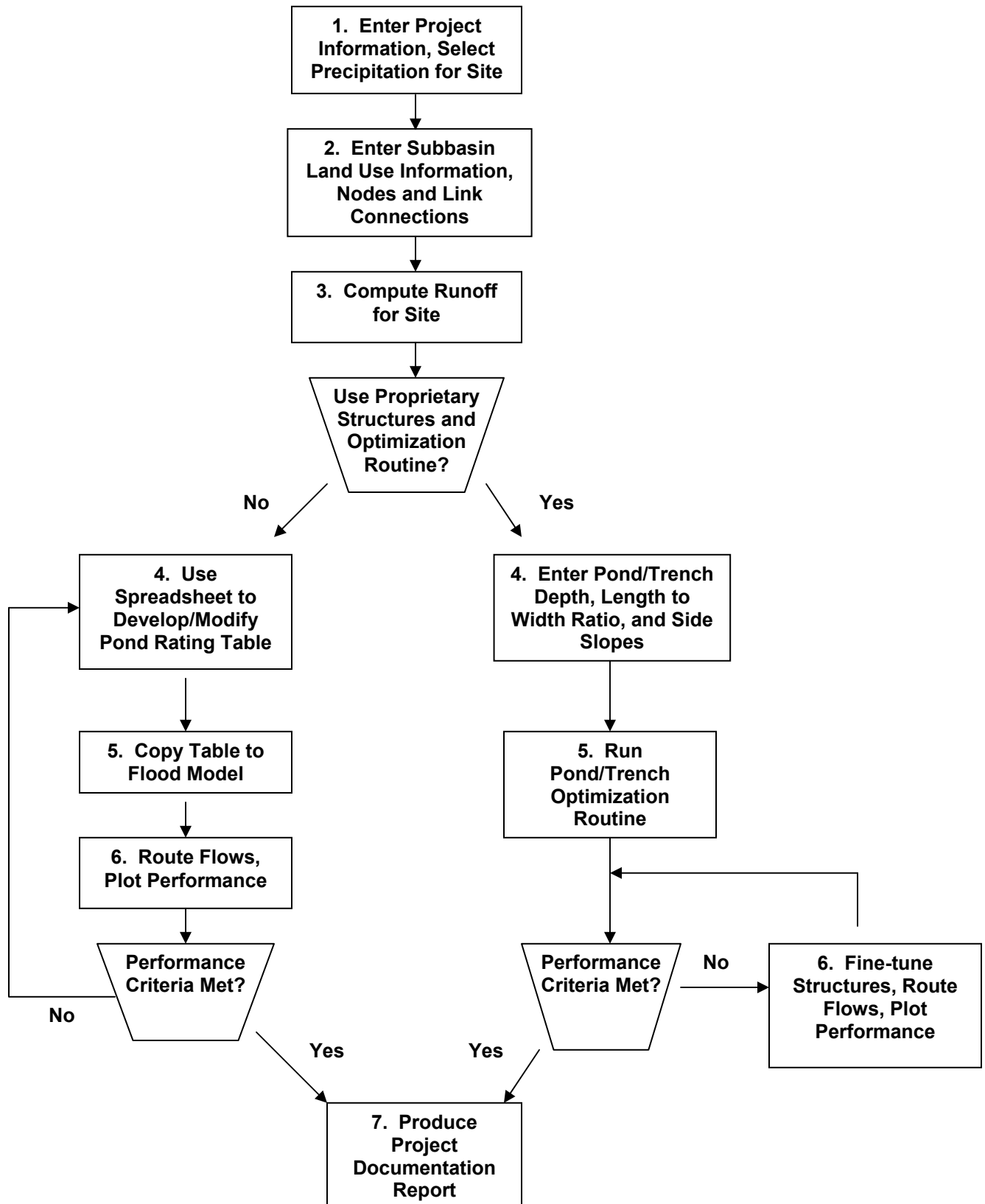


Figure 3.1 – MGSFlood Tabs at Bottom of Input Screen

A flow chart showing the stormwater pond design process is shown in Figure 3.2. The sequence of steps to design a stormwater pond depends on whether the hydraulic structures and optimization routine is used. If the hydraulic structures and optimization routines are not used, then a spreadsheet or other program must be used to compute a hydraulic rating table for input to MGSFlood. Using this method, the pond is designed iteratively where pond configurations developed in the spreadsheet are tested in MGSFlood.

If the optimization routine is used, then the pond and outlet works are determined automatically by the program, with only minor adjustments typically needed to refine the design determined by the program.

**Figure 3.2 - Stormwater Pond Design Procedure Using MGS Flood**



## 4 Starting Program, Saving Data

MGSFlood is installed to a default folder in the \Program Files directory. A shortcut created under the Start menu in the *Programs-MGSSoftware* folder can be used to start the program. Graphics Server is a graphics package used by MGSFlood to plot statistics and hydrographs and is installed with MGSFlood. When MGSFlood terminates, Graphics Server is unloaded from memory.

MGSFlood creates a number of files on disk for each project so it is recommended that a separate folder be created for each project. This can be accomplished automatically when saving the project for the first time. The program will prompt for the creation of a new folder with the project name (Figure 4.1). Responding yes to this prompt will create a new folder with the project file stored in it. All subsequent files created by the program for the project will be stored in this directory.

The default directory for saving data files can be set from the View menu. The default directory can be any directory mapped to the computer, including network drives.

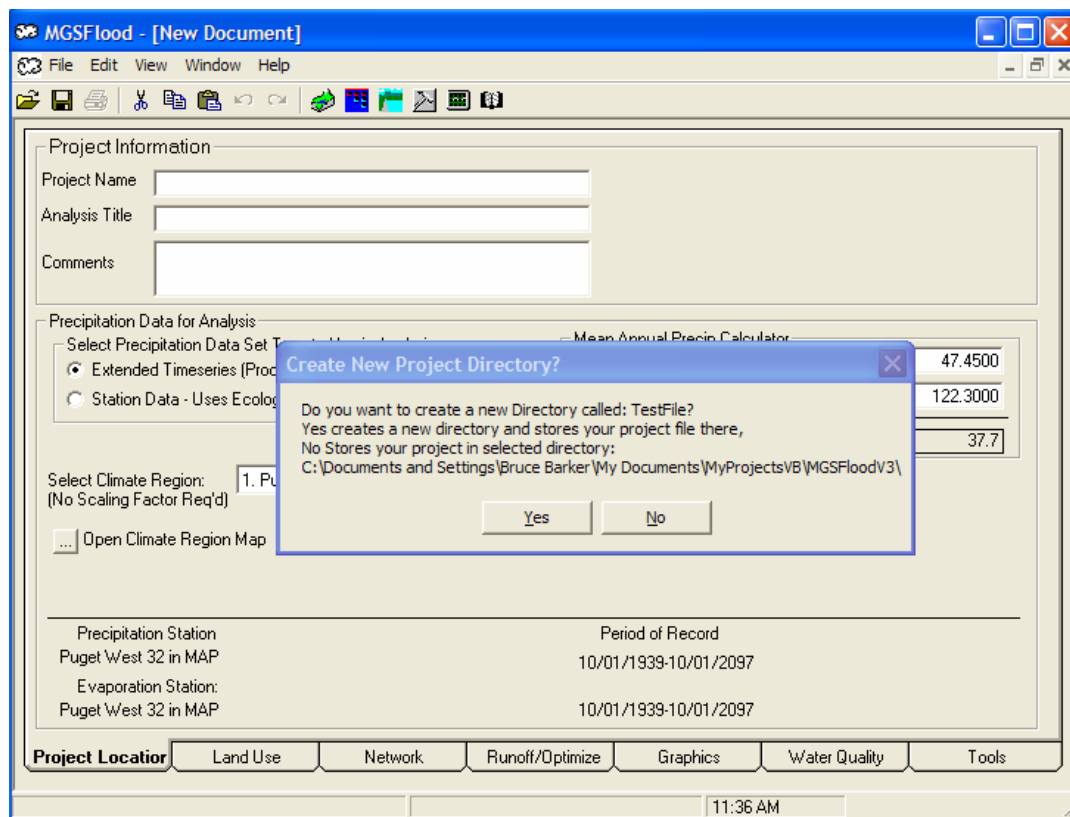


Figure 4.1 – Prompt to Create a new Project Folder when Saving a Project for the First Time

## 5 Getting Help

Context sensitive help is available by pressing F1 or by selecting Help from the command menu at the top of the screen.

## 6 Project Location Tab

The project location tab contains two different types of data; *Project Information* and *Precipitation Data Used in Analysis* (Figure 6.1). Data fields in the Project Information section are used for identifying the project. Information entered here is printed on the project reports.

The program contains two options for selecting precipitation input for project analysis; *Extended Precipitation Timeseries* and *Station Data*. The two options are discussed in the following sections.

MGSFlood - [TestFile.fld]

File Edit View Window Help

Project Information

Project Name

Analysis Title

Comments

Precipitation Data for Analysis

Select Precipitation Data Set Type to Use in Analysis

☒ Extended Timeseries (Produces Most Accurate Results)

☐ Station Data - Uses Ecology Scaling Method

Mean Annual Precip Calculator

Project Latitude (Decimal Degrees): 47.4500

Project Longitude (Decimal Degrees): 122.3000

... Compute MAP (inches) 37.7

Select Climate Region: 1. Puget West 32 in MAP (No Scaling Factor Req'd)

... Open Climate Region Map

Precipitation Station	Period of Record
Puget West 32 in MAP	10/01/1939-10/01/2097
Evaporation Station:	
Puget West 32 in MAP	10/01/1939-10/01/2097

Project Location Land Use Network Runoff/Optimize Graphics Water Quality Tools

11:41 AM

Figure 6.1 – Project Location Tab

## 6.1 Extended Precipitation Timeseries Selection

Extended Precipitation timeseries utilizes a family of pre-scaled precipitation and evaporation timeseries. These timeseries were developed by combining and scaling precipitation records from widely separated stations resulting in record lengths in excess of 100-years. Extended hourly precipitation and evaporation timeseries have been developed using this method for most of the lowland areas of western Washington where stormwater projects will be constructed. These timeseries should be used for facility design for projects located in the region shown in Figure 6.2.

To select the precipitation and evaporation input for a project, open the Precipitation Map from the Project Location Tab. Locate the project site on the map and note the zone and the mean annual precipitation for the project site. The mean annual precipitation may also be determined by entering the project latitude and longitude in the Mean Annual Precipitation Calculator (in decimal degrees) and clicking the *Compute MAP* button.

From the Climatic Region drop down box on the *Project Location* tab, select the precipitation timeseries corresponding to the region and mean annual precipitation noted from the map or computed from the calculator. For project sites located in Pierce County, select the Pierce County precipitation timeseries corresponding to the region and mean annual precipitation for the project site.

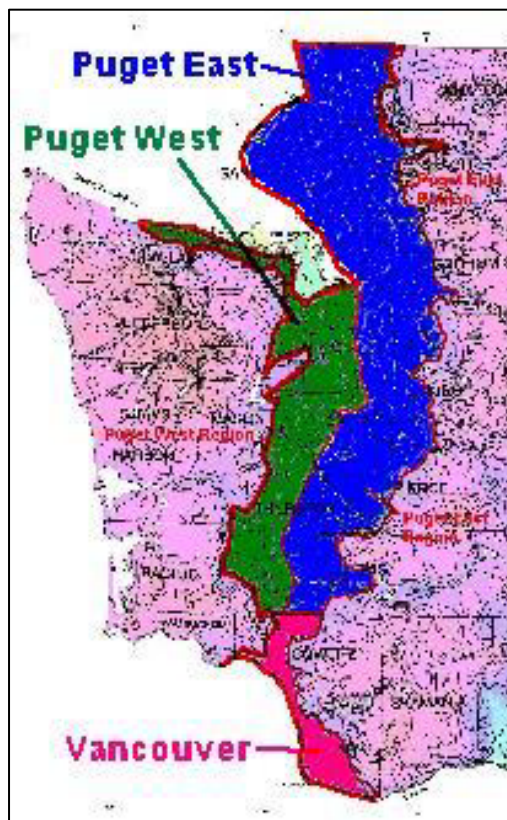


Figure 6.2 – Extended Precipitation Timeseries Regions

The example project site shown in Figure 6.3 is located in the western Puget Sound Region and the project mean annual precipitation is 51 inches. The precipitation timeseries for the western Puget Sound Region with mean annual precipitation closest to the project site should be selected from the drop down box. In this case, Puget Sound West Region, 52 inches MAP should be used.



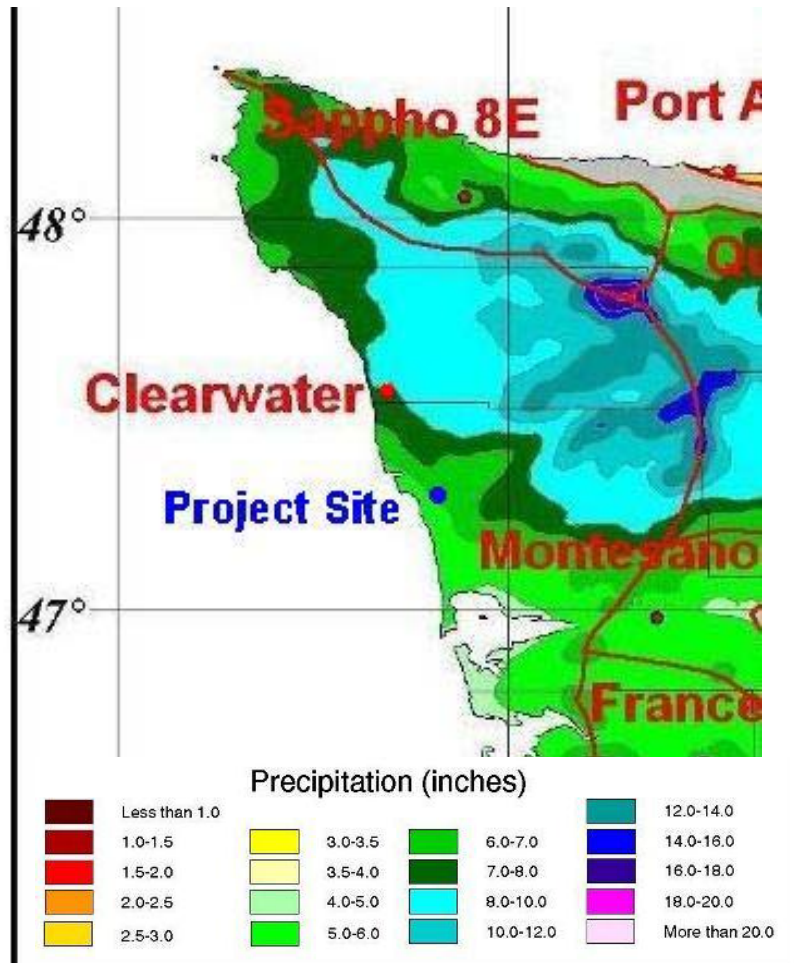
Figure 6.3 – Extended Precipitation Timeseries Selection Example

## 6.2 Precipitation Station Selection

For projects sites located outside of the extended timeseries region, a *source* gage is selected and a single scaling factor is applied to transpose the hourly record to the site of interest (target site). The current approach for single factor scaling, as recommended in the *Stormwater Management Manual for Western Washington*<sup>9</sup>, is to compute the scaling factor as the ratio of the 25-year 24-hour precipitation<sup>21</sup> for the target and source sites.

To select the precipitation and evaporation input for a project location outside the area where the extended precipitation timeseries apply, check the *Station Data* option button and open the Precipitation Map from the Project Location Tab. Choose the precipitation region where the project site is located. Read the project site 25-year 24-hour precipitation from the map and enter it in the appropriate field on the Project Location Tab. The project 25-year 24-hour precipitation may also be computed by entering the project latitude and longitude in the Precip Calculator (in decimal degrees) and clicking the *Compute 25-Yr. 24-Hr* button.

For the example project site shown in Figure 6.4, the Clearwater gage should be selected as the source gage, and a project site 25-year, 24-hour precipitation of 6.0 inches should be entered in the appropriate field on the Project Location tab. The Scale factor would be computed by the program as the ratio of the project site to station 25-year, 24-hour precipitation, or 6.0 inches divided by 7.9 inches equals 0.759 (MGSFlood limits the scale factor to a minimum of 0.80 and no constraint on the maximum scaling factor is imposed). This value would be displayed in the *Scale Factor* field and all precipitation values subsequently read by the program would be multiplied by this value.



**Figure 6.4 – Precipitation Input Selection Example for Project Sites Located Outside of Region Covered by Extended Timeseries**

## 7 Land Use Tab

Subbasin areas and how the areas are connected to nodes are defined on the *Land Use* Tab.

### 7.1 Subbasin Areas

Land use for each subbasin is defined on the *Land Use* tab (Figure 7.1). Up to six subbasins can be defined; however, the program is currently being modified to allow any number of subbasins.

Watershed Area (Acres)	Developed		
	Predeveloped	Tributary to Node	By-Pass Node
Till Forest	0.000	0.000	0.000
Till Pasture	0.000	0.000	0.000
Till Grass	0.000	0.000	0.000
Outwash Forest	0.000	0.000	0.000
Outwash Pasture	0.000	0.000	0.000
Outwash Grass	0.000	0.000	0.000
Wetland	0.000	0.000	0.000
User	0.000	0.000	0.000
User	0.000	0.000	0.000
Lateral 1	0.000	0.000	0.000
Lateral 2	0.000	0.000	0.000
Impervious	0.000	0.000	0.000
<b>Total (acres)</b>	<b>0.000</b>	<b>0.000</b>	<b>0.000</b>

Node Connections

Connect Subbasin to Node: Node 1

Connect By-Pass Area to Node: None

Lateral Flow Connection Option

Predeveloped Lateral Flow/ Flow Dispersion

Developed Lateral Flow/ Flow Dispersion

Click to view Potential Regulatory Restrictions Regarding Land Use Input

Project Location Land Use Network Runoff/Optimize Graphics Water Quality Tools

11:46 AM

Figure 7.1 – Land Use Definition Screen

Consult the stormwater management manual for the local regulatory jurisdiction and the Washington State Stormwater Management Manual for Western Washington<sup>9</sup> (SWMMWW) regarding possible regulatory restrictions for:

Predeveloped Forest Cover - There are restrictions concerning the designation of the predeveloped land use as anything other than forest (SWMMWW Volume I, Minimum Requirement 7),

Post Developed Forest Or Pasture Cover - Assurances are required when designating an area as forest or pasture for the postdevelopment state to ensure that the area will not be disturbed in the future (SWMMWW Volume III, Appendix B),

Off-Site Run-On To Project - There are limits to offsite inflow discharging to a stormwater detention facility (SWMMWW Volume III, Appendix B),

On-Site Stormwater Bypass - There are restrictions to the size of development area from which stormwater runoff may bypass a detention facility (SWMMWW Volume III, Appendix B).

Regulatory restrictions regarding land use input may also be viewed by clicking the button on the Subbasin Area Definition Screen.

## 7.2 Area Tributary to Node

For each subbasin, land use is defined in acres for *Predeveloped* and *Developed* conditions. The developed conditions entry includes a column for the number of acres tributary to and the number of acres that bypass the node (Figure 7.1). The *Stormwater Management Manual for Western Washington*<sup>9</sup> relates SCS hydrologic soil groups to HSPF soil/geologic groups as shown in Table 7.1

**Table 7.1 – Relationship between SCS and HSPF Soil Groups**

SCS Hydrologic Soil Group	MGSFlood/HSPF Soil/Geologic Group
A/B	Outwash
C	Till
D	Wetland

Note: The surface area of the pond must be included under the land use for the subbasin because precipitation is not applied to the pond surface by the program. This can be accomplished by adding impervious surface equal to the maximum pond surface area under the Subbasin Definitions window.

## 7.3 Area Bypassing Node

Local topographic constraints often make it impractical to direct all runoff from developed areas to a detention facility. The bypass feature allows an area of the subbasin to be connected to a node downstream of the stormwater pond as shown in Figure 7.2. In this example, Node 2 becomes the point of compliance for the pond meaning that the postdevelopment flows at Node 2 would have to be controlled to the predeveloped level computed at Node 1.

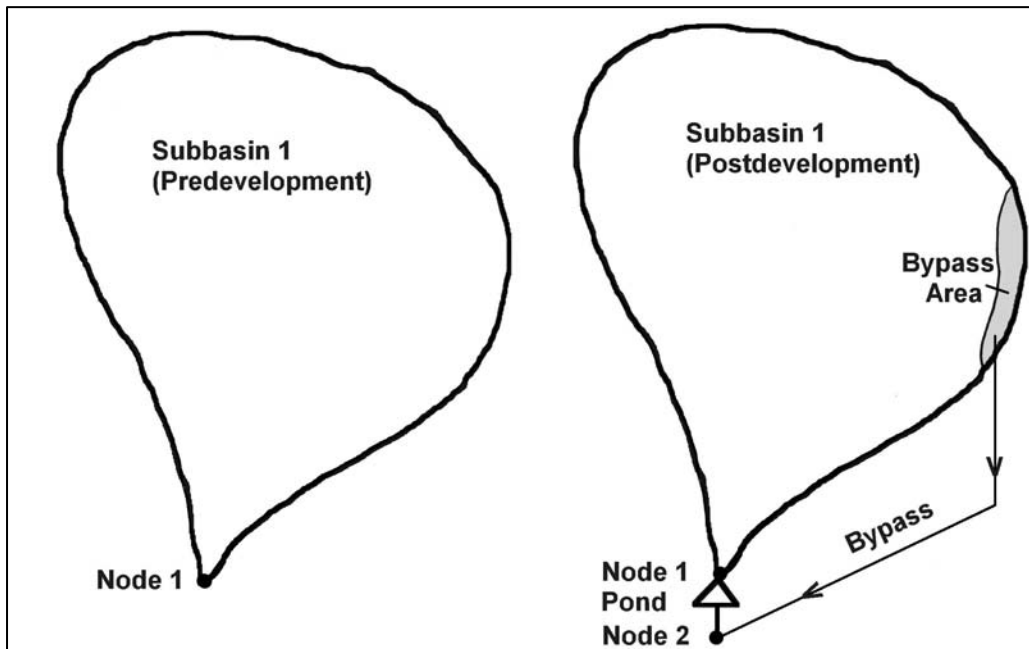


Figure 7.2 – Subbasin and Node Delineation, Single Subbasin with Bypass

#### 7.4 Lateral Flow Connections

MGSFlood simulates runoff from pervious areas as three components; surface overland flow, interflow, and groundwater flow. The *Lateral Flow Connection Option* button can be used to define which runoff components are connected to the receiving node. By default, surface and interflow are connected to the subbasin receiving node with the groundwater component omitted (Figure 7.3). In cases where high groundwater levels results in discharge to the drainage system, the groundwater component may be included in the runoff calculations. Similarly, any of the runoff components can be included or omitted for any of the land cover/soil types in the project.

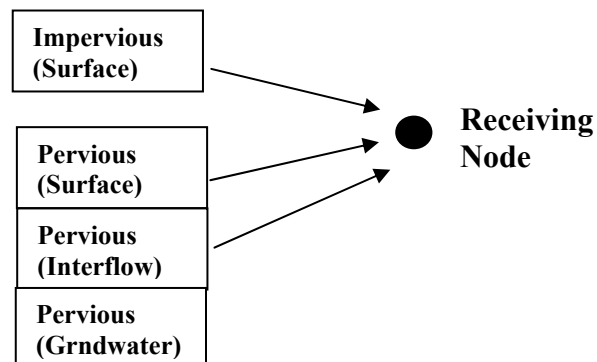


Figure 7.3 – Default Runoff Components (Surface and Interflow) Connected to Subbasin Receiving Node

Runoff from upslope areas may also be connected to downslope pervious areas to simulate surface runoff dispersion and infiltration. Figure 7.4 shows runoff from an area of impervious surface that is dispersed over a pervious area before reaching the receiving stream. The infiltration of the impervious runoff would be simulated in the model by connecting the impervious surface runoff to the downslope Lateral Flow PerInd. Each subbasin has two separate Lateral Flow PerInds that can be defined to simulate flow dispersion. Upslope surface flow, interflow, and groundwater may be connected to the downslope surface, interflow, or groundwater flow components. Using HSPF nomenclature, the upslope components represent SURO, IFWO, and AGWO which may be connected to the downslope SURLI, IFWLI, or AGWLI.

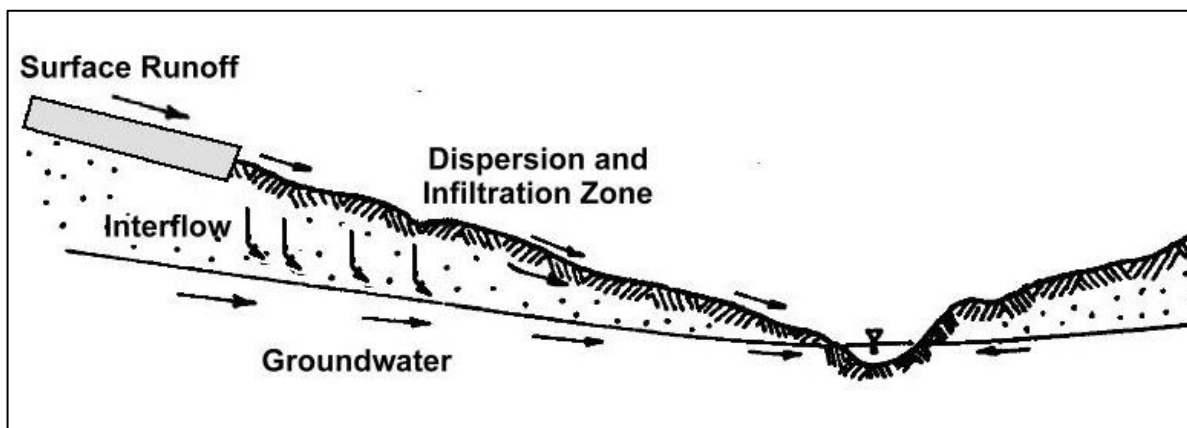


Figure 7.4 – Surface Runoff Dispersion and Infiltration from Upslope Impervious Surface over Downslope Pervious Surface

Figure 7.5 shows an example schematic where runoff from impervious surface and surface and interflow from till/grass are connected to a downslope pervious area. The upslope surface components are connected to the downslope surface component, the upslope interflow component is connected to the downslope interflow component, and the downslope surface and interflow components are connected to the receiving node. A second downslope pervious land segment may also be added downstream of the first to account for different soil types in the dispersion area.

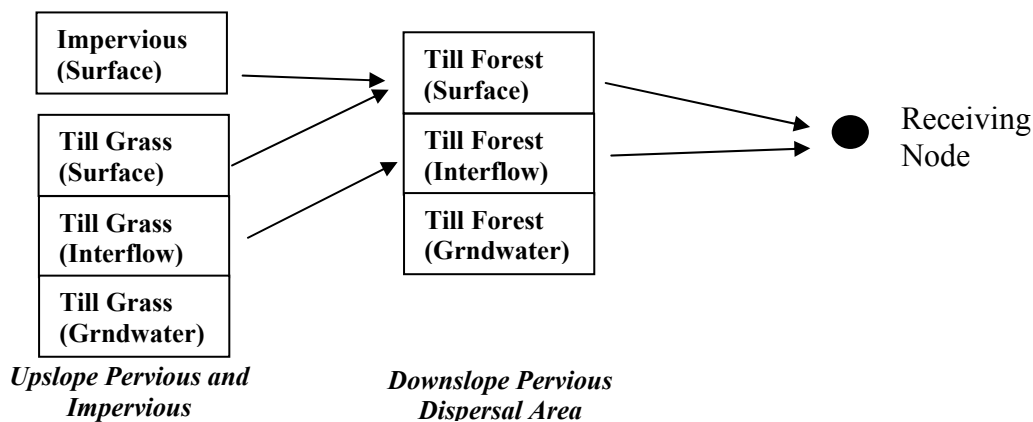


Figure 7.5 – Example Flow Dispersion Schematic, Upslope Impervious and Grass Routed to Downslope Till Forest Area (Note, only surface and interflow connected in this example)

Figure 7.6 shows the input screen in the MGSFlood model for this example. Note that the runoff parameters for Lat 1 (the downslope dispersion and infiltration zone) are defined in the list box in the lower right of the screen.

Upstream Segment	Connect to:	Surface to:	Interflow to:	Groundwater to:	Lateral Flow Runoff Parameters
Till Forest	Node	Node	Node		
Till Pasture	Node	Node	Node		
Till Grass	Lat 1	Surface	Interflow		
Outwash Forest	Node	Node	Node		
Outwash Pasture	Node	Node	Node		
Outwash Grass	Node	Node	Node		
Wetland	Node	Node	Node		
User1	Node	Node	Node		
User2	Node	Node	Node		
Impervious	Lat 1	Surface			
Lateral 1	Node	Node	Node		Till Forest
Lateral 2	Node	Node	Node		Till Forest

**Figure 7.6 – Upslope Impervious and Till Grass Connected to Downslope Till Forest, Surface and Interflow Only**  
**Downslope Till Forest Surface and Interflow are Connected to the Receiving Node**

## 7.5 High Groundwater Simulation

The model includes the high groundwater simulation option present in HSPF Version 12<sup>26</sup>. This routine was designed for use with soils that can be influenced by seasonal high groundwater that approaches or rises above the ground surface. This method, which is applicable for wetlands and low gradient areas, is a modified version of the standard hydrologic method in HSPF (i.e., the Stanford Watershed Model). The routine converts the conceptual storages in the HSPF PERLND routine to groundwater elevation above an arbitrary datum using soil properties such as porosity. The following sections briefly describe the routine and the user is referred to the HSPF Version 12 manual for more information.

### 7.5.1 High Groundwater Simulation Soil Properties

The input screens for the High Groundwater Simulation option are accessed by clicking the *Lateral Flow Connection Option* Button from the *Land Use* Tab, then clicking the *High Groundwater* Tab (Figure 7.7). Any of the active land segments in the project may be

defined as being influenced by high groundwater by checking the Enable High GW box for the land segment of interest. Parameters defining the soil properties are entered on the *High Groundwater 1* and *High Groundwater 2* tabs.

**Lateral Flow/High Groundwater Definitions**

High Groundwater Parameters

Land Segment	High Gw Enable	Ground	Min Channel	Datum	PCW	PGW	UPGW	IFW/SC	Delta	UELFAC	LELFAC
		Elev (ft)	Elev (ft)	Elev (ft)	Porosity	Porosity	Porosity	(inches)	(inches)		
Till Forest	<input type="checkbox"/>	100.00	99.00	95.00	0.30	0.60	0.60	4.50	0.001	4.00	2.50
Till Pasture	<input type="checkbox"/>	100.00	99.00	95.00	0.30	0.60	0.60	4.50	0.001	4.00	2.50
Till Grass	<input checked="" type="checkbox"/>	100.00	99.00	95.00	0.30	0.60	0.60	4.50	0.001	4.00	2.50
Outwash Forest	<input type="checkbox"/>	100.00	99.00	95.00	0.30	0.60	0.60	4.50	0.001	4.00	2.50
Outwash Past	<input type="checkbox"/>	100.00	99.00	95.00	0.30	0.60	0.60	4.50	0.001	4.00	2.50
Outwash Grass	<input type="checkbox"/>	100.00	99.00	95.00	0.30	0.60	0.60	4.50	0.001	4.00	2.50
Wetland	<input type="checkbox"/>	100.00	99.00	95.00	0.30	0.60	0.60	4.50	0.001	4.00	2.50
User1	<input type="checkbox"/>	100.00	99.00	95.00	0.30	0.60	0.60	4.50	0.001	4.00	2.50
User2	<input type="checkbox"/>	100.00	99.00	95.00	0.30	0.60	0.60	4.50	0.001	4.00	2.50
Lateral 1	<input type="checkbox"/>	100.00	99.00	95.00	0.30	0.60	0.60	4.50	0.001	4.00	2.50
Lateral 2	<input type="checkbox"/>	100.00	99.00	95.00	0.30	0.60	0.60	4.50	0.001	4.00	2.50

Ok Cancel

**Figure 7.7 – High Groundwater Input Screen**

For modeling purposes, the total porosity is divided into porosity in micropores (PCW, cohesion water), and porosity in macropores (PGW, gravitational water). The upper layer of the soil may be disturbed and have a larger porosity in macropores (UPGW). The porosity of micropores is assumed to be the same throughout the soil column. Cohesion water is stored in the lower zone storage, while gravitational water is stored in the upper zone and interflow storages.

Pcw is the porosity of cohesion water and Pgw is the porosity of gravity water (Figures 7.8a and 7.8b).

The “groundwater level” is the elevation of the saturated zone above an arbitrary datum such as mean sea level. The active groundwater storage is gravity water stored above the minimum channel or canal elevation that is within or adjacent to the land.

The three soil regions in the soil column are determined by the lower influence elevation (LELV) and the upper influence elevation (UELV).

The lower influence elevation is the elevation above which the groundwater affects lower zone behavior and, depending on the current interflow storage, it may also affect interflow behavior.

Ground Elev - is the mean surface elevation of the land segment (ft).

Min Channel Elevation - is the base elevation for active groundwater. It corresponds to the bottom elevation of nearby channels; therefore, if the groundwater elevation is above BELV, there is outflow into the channels. Groundwater below BELV is considered inactive.

GWDATM - is the datum for the groundwater elevation GWEL (ft). Storage below this elevation is considered lost from the system.

PCW - is the cohesion water porosity. It is the soil pore space in micropores.

PGW - is the gravitational water porosity. It is the soil pore space in macropores in the lower and groundwater layers of the soil column.

UPGW - is the upper gravitational water porosity. It is the pore space in macropores in the upper layers of the soil column.

IFWSC - is the maximum interflow storage capacity when the groundwater elevation is greater than the upper influence elevation (UELIV).

DELTA - is the groundwater tolerance level used to determine transition between regions when high water table conditions are being simulated. It is used to smooth out jumps in groundwater elevation due to changes in "soil region." The default value should generally be used.

UELFAC - is the multiplier on UZSN which is used to compute the upper zone capacity. The default value (4.0) should generally be used.

LELFAC - is the multiplier on LZSN which is used to compute the lower zone capacity. The default value (2.5) should generally be used.

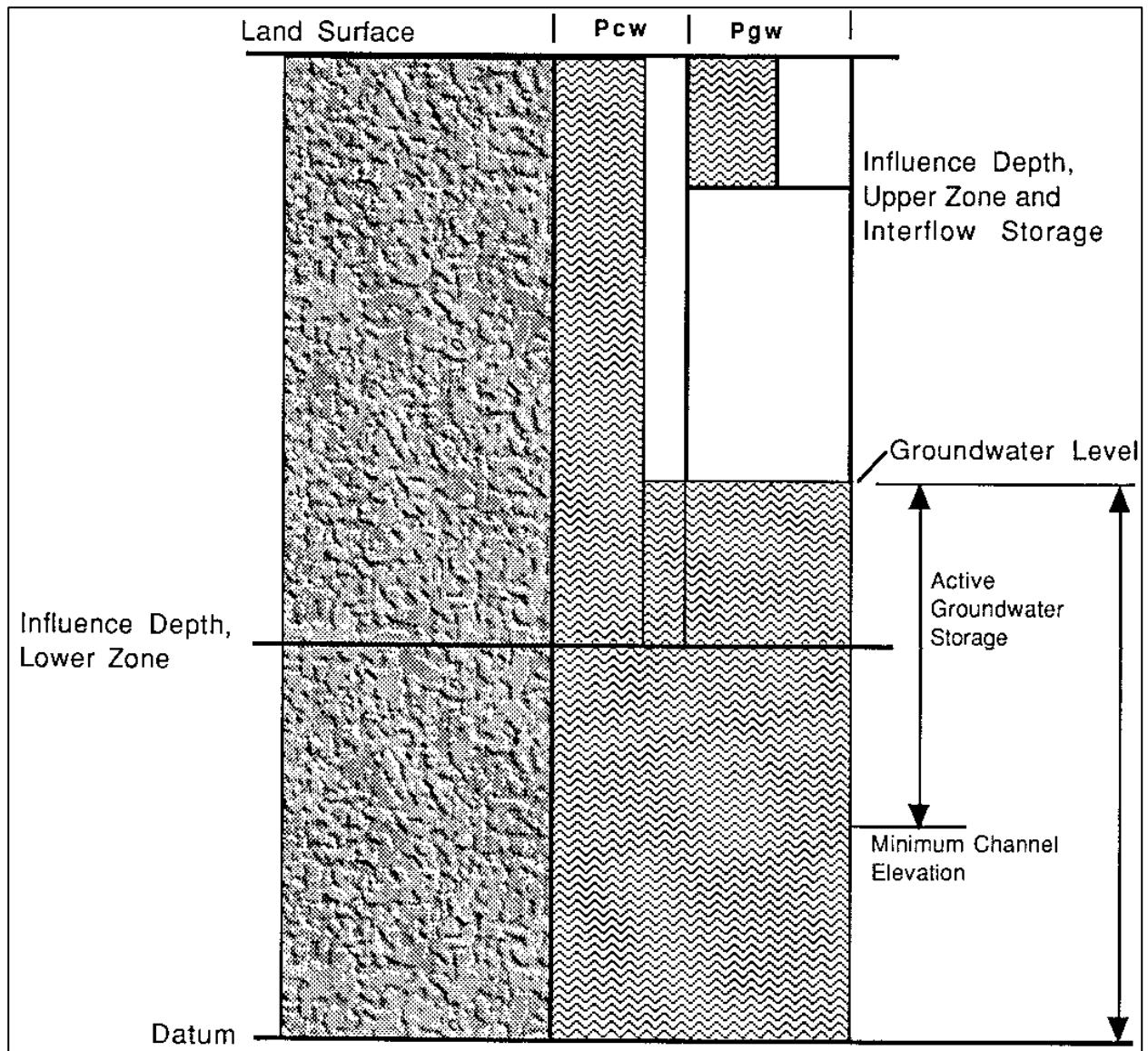


Figure 7.8 a – Sketch of Soil Moisture in the Unsaturated Zone Under High Water Table Conditions

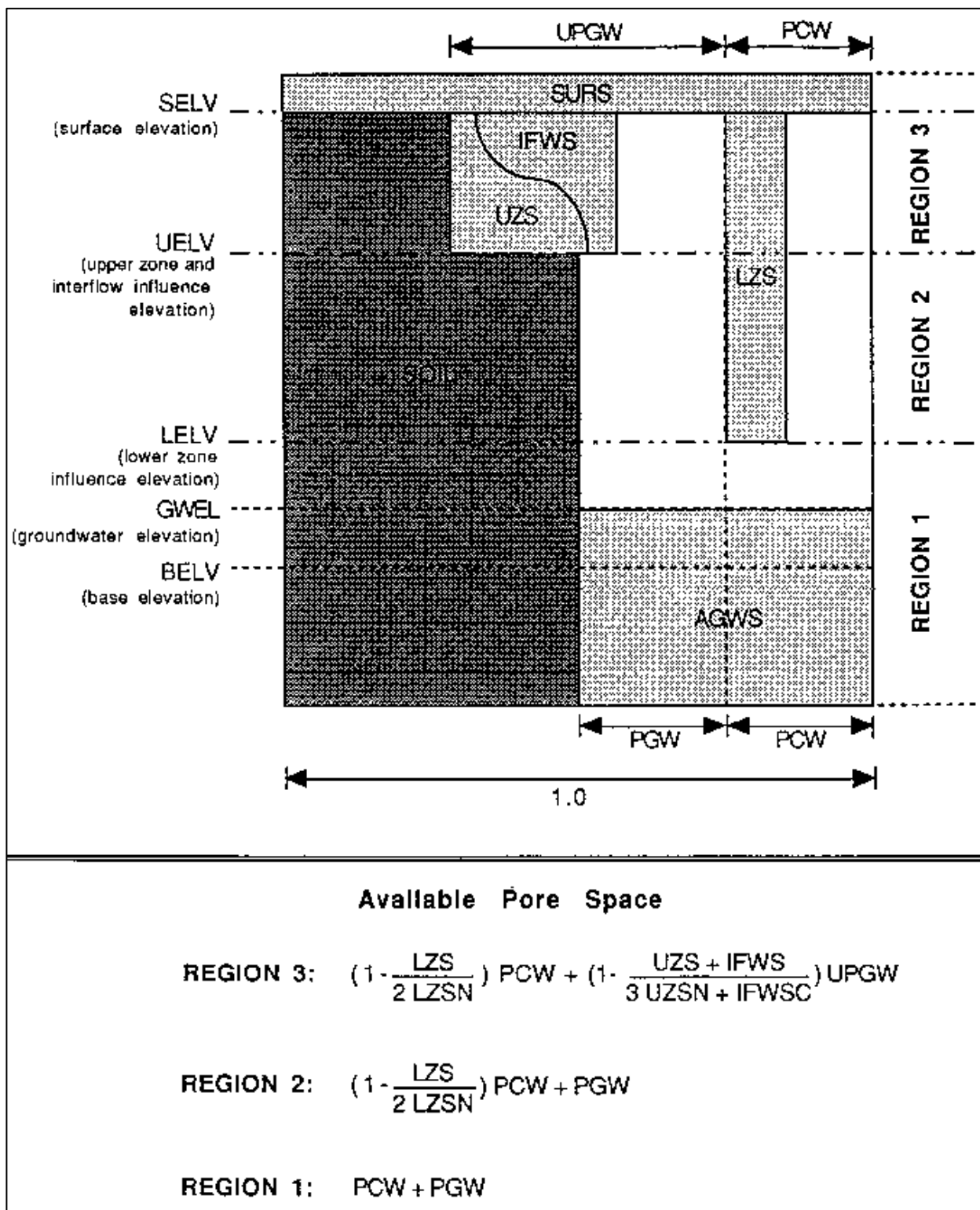


Figure 7.8 b – Soil moisture storage concepts under high water table conditions

### 7.5.2 High Groundwater Simulation Surface Ponding Definition

When the groundwater level reaches the surface, any additional water is assigned to surface detention storage and the surface detention storage represents the depth of inundation over the land segment. Evaporation from the surface storage takes place at the potential rate after interception storage has been exhausted and runoff is defined based on the surface hydraulics of the wetland.

Input screens that define the surface runoff hydraulics are accessed from the *High Groundwater 2* tab. The wetland storage is entered as an elevation, surface area, volume table (Figure 7.9). Information may be copied to this table using the Windows Clipboard utility.

**High Groundwater PerInd, Surface Ponding Definition**

Edit

**Elevation/Volume** Discharge

**Surface Ponding, Elevation, Surface Area, Volume Relationship**

Elevation-Surface Area-Volume Values  
Must be Increasing

Row	Elev (ft)	Surf Area (sf)	Volume (cu-ft)
1	100.00	0.0	0.00
2	101.00	43560.0	43560.00
3	102.00	43560.0	87120.00
4	103.00	43560.0	130680.00
5			
6			
7			
8			
9			
10			
11			
12			
13			
14			
15			

Note: The Surface Area is Typically Entered as the Area Defined on the Subbasin Area Tab.  
**Area Entered on Subbasin Tab (sf): 43,560.**

Ok Cancel

Figure 7.9 – Input Screen for Defining Wetland Storage

Wetland discharge characteristics are defined using structures available for stormwater ponds, including weirs, orifices, riser structures, etc. Any combination of outlet structures may be defined. For cases where the outlet is controlled by an open channel, the trapezoidal weir should be used. This has been set as the default outlet structure (Figure 7.10).

**High Groundwater PerInd, Surface Ponding Definition**

Edit

Elevation/Volume **Discharge**

**Outlet Structure(s)**

Enable	Structure Type	Crest El. (ft)	Length (ft)	Side Slope (Z)
<input checked="" type="checkbox"/>	Trapezoidal Broad Crest Weir	100.00	2.000	3.00
<input type="checkbox"/>	Circular Orifice			
<input type="checkbox"/>	Circular Orifice			
<input type="checkbox"/>	Circular Orifice			
<input type="checkbox"/>	Circular Orifice			
<input type="checkbox"/>	Circular Orifice			

Orientation Elbow

☒ Horizontal ☒ Yes

☐ Vertical ☐ No

Orientation Elbow

☒ Horizontal ☒ Yes

☐ Vertical ☐ No

Orientation Elbow

☒ Horizontal ☒ Yes

☐ Vertical ☐ No

Orientation Elbow

☒ Horizontal ☒ Yes

☐ Vertical ☐ No

Orientation Elbow

☒ Horizontal ☒ Yes

☐ Vertical ☐ No

**Riser Structure**

Enable	Structure Type	Crest El. (ft)	Diameter (in)	Common L (ft)	Riser Top Open
<input type="checkbox"/>	Circular Overflow Riser				<input checked="" type="radio"/> Yes <input type="radio"/> No

Ok Cancel

Figure 7.10 – Input Screen for Defining Wetland Outlet Hydraulics

## 8 Network Tab

Nodes are used to collect runoff from the tributary area for a given subbasin and from the nodes of upstream subbasins. Links are used to connect one node to another and may be defined as one of the following:

1. Copy – Copies discharge from upstream node and adds it to discharge at the downstream node without routing,
2. Structure – Includes detention and infiltration ponds, and sand filters,
3. Channel – Performs routing in open channels,
4. Infiltration Trench – Performs routing through infiltration trenches,
5. Rating Table – User defined stage storage discharge table,
6. Flow Splitter – Splits a fraction of the discharge from one node to another.

Currently, the program is configured to accommodate a maximum of 10 nodes with 9 links. Separate predeveloped and postdeveloped network configurations are defined to accommodate the sizing of stormwater treatment facilities in the post-developed condition. Information for each type of Link is discussed in the following Sections.

### 8.1 Predeveloped and Postdeveloped Network Configurations

The program allows for different network configurations for predeveloped and postdeveloped conditions. This allows for structures, such as stormwater ponds, to be added in the post-developed condition without having to specify a corresponding dummy reach in the predeveloped condition.

Network configurations are defined by specifying the upstream node, node name, downstream node, and the type of link that connects them. Parameters associated with each link are input by clicking the *Link Definition* button at the right side of the screen (Figure 8.1). The *Compliance* option button indicates the predevelopment point of compliance. The program will use this node along with the postdeveloped compliance node to determine flow control compliance.

The postdeveloped network configuration is specified on the *Proposed Condition* tab similar to the predeveloped. The program will automatically determine the size of pond or infiltration trench if the *Optimize* check box is selected (Figure 8.2). Only ponds and infiltration trenches may be sized with the automatic routine. When the Optimize box is checked, the node at the downstream end of the link to be optimized becomes the point of compliance.

Information and input parameters for each type of link selected is input by clicking the *Link Definition* button for the link.

**Existing Condition**

Upstream Node	Upstream Node Name	Compliance Point	Downstream Node	Link Type	Link Definition
Node 1	Node 1	<input type="radio"/>	Node 2	Channel Rot	...
Node 2	Node 2	<input checked="" type="radio"/>	None	Copy	...
Node 3	Node 3	<input type="radio"/>	None	Copy	...
Node 4	Node 4	<input type="radio"/>	None	Copy	...
Node 5	Node 5	<input type="radio"/>	None	Copy	...
Node 6	Node 6	<input type="radio"/>	None	Copy	...
Node 7	Node 7	<input type="radio"/>	None	Copy	...
Node 8	Node 8	<input type="radio"/>	None	Copy	...
Node 9	Node 9	<input type="radio"/>	None	Copy	...
Node 10	Node 10	<input type="radio"/>	None	Copy	...

Watershed Schematic

Project Location Land Use **Network** Runoff/Optimize Graphics Water Quality Tools

1:50 PM

**Figure 8.1 – Network Definition Input, Predeveloped Condition**

**Proposed Condition**

Upstream Node	Upstream Node Name	Compliance Point	Downstream Node	Link Type	Optimize	Link Definition
Node 1	Node 1		Node 2	Structure	<input checked="" type="checkbox"/>	...
Node 2	Node 2		None	Copy	<input type="checkbox"/>	...
Node 3	Node 3		None	Copy	<input type="checkbox"/>	...
Node 4	Node 4		None	Copy	<input type="checkbox"/>	...
Node 5	Node 5		None	Copy	<input type="checkbox"/>	...
Node 6	Node 6		None	Copy	<input type="checkbox"/>	...
Node 7	Node 7		None	Copy	<input type="checkbox"/>	...
Node 8	Node 8		None	Copy	<input type="checkbox"/>	...
Node 9	Node 9		None	Copy	<input type="checkbox"/>	...
Node 10	Node 10		None	Copy	<input type="checkbox"/>	...

Watershed Schematic

Project Location Land Use **Network** Runoff/Optimize Graphics Water Quality Tools

3:29 PM

**Figure 8.2 – Network Definition Input, Postdeveloped Condition**  
 (The Pond Link Connecting Node 1 and Node 2 will be Automatically Sized using the Optimization Routine)

## 8.2 Example Network Configuration

Node connections allow the user to connect runoff from multiple subbasins at a single point or connect bypass flows to a location downstream of the stormwater pond. In the example shown in Figure 8.3, Subbasins 1, 2, and 3 are connected to Nodes 1, 2, and 3 respectively, which represent the runoff from each subbasin. The runoff from each subbasin is copied to Node 4. Node 4 represents the inflow to a stormwater pond. Two acres of Subbasin 3 bypasses the pond to Node 5. Runoff from Node 4 will be used as input for sizing the stormwater detention pond with Node 5 used to save routed flows from the pond. The bypassed flows from Subbasin 3 will be added to the pond outflow during routing, and Node 5 will be the point of compliance. Following routing computations, flows stored at Node 5 will be the sum of the routed flows and the bypassed flows from Subbasin 3. Note that flows from bypassed areas are added to the downstream node without routing.

The predeveloped network input screen is shown in Figure 8.4. Note that Node 4 is the predeveloped point of compliance. The postdeveloped network input is shown in Figure 8.5 with Node 5 being the point of compliance because of the stormwater pond to be constructed to mitigate runoff. The optimization routine automatically assigns node downstream of the link being optimized as the compliance point.

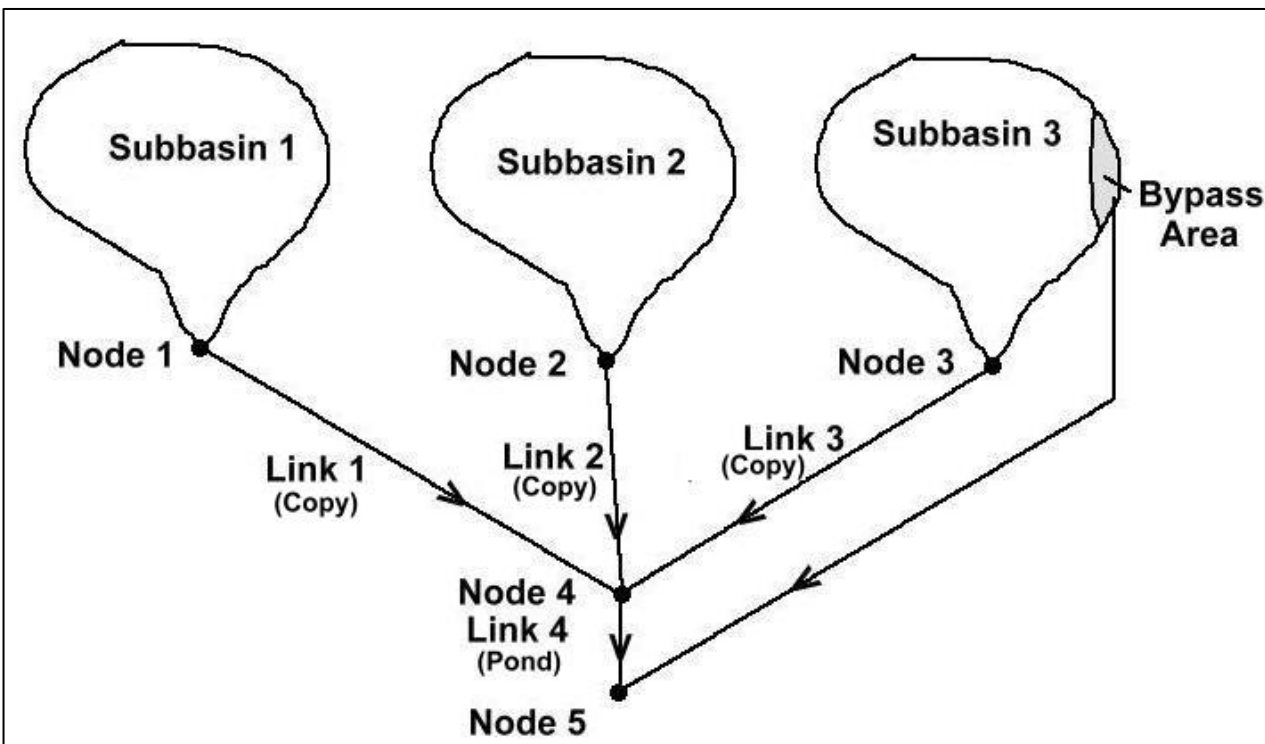


Figure 8.3 – Subbasin, Node, and Link Delineation, Multiple Subbasin Example with Bypass

MGSFlood - [MultipleBasin.fld]

File Edit View Window Help

Existing Condition Proposed Condition

Upstream Node	Upstream Node Name	Compliance Point	Downstream Node	Link Type	Link Definition
Node 1	Subbasin 1	<input type="radio"/>	Node 4	Copy	...
Node 2	Subbasin 2	<input type="radio"/>	Node 4	Copy	...
Node 3	Subbasin 3	<input type="radio"/>	Node 4	Copy	...
Node 4	Point of Compliance	<input checked="" type="radio"/>	None	Copy	...
Node 5	Node 5	<input type="radio"/>	None	Copy	...
Node 6	Node 6	<input type="radio"/>	None	Copy	...
Node 7	Node 7	<input type="radio"/>	None	Copy	...
Node 8	Node 8	<input type="radio"/>	None	Copy	...
Node 9	Node 9	<input type="radio"/>	None	Copy	...
Node 10	Node 10	<input type="radio"/>	None	Copy	...

Watershed Schematic

Project Location Land Use **Network** Runoff/Optimize Graphics Water Quality Tools

3:45 PM

Figure 8.4 – Predevelopment Network Input Screen for Subbasin/Node Connection Example

MGSFlood - [MultipleBasin.fld]

File Edit View Window Help

Existing Condition Proposed Condition

Upstream Node	Upstream Node Name	Compliance Point	Downstream Node	Link Type	Optimize	Link Definition
Node 1	Subbasin 1		Node 4	Copy	<input type="checkbox"/>	...
Node 2	Subbasin 2		Node 4	Copy	<input type="checkbox"/>	...
Node 3	Subbasin 3		Node 4	Copy	<input type="checkbox"/>	...
Node 4	Pond Inflow		Node 5	Structure	<input checked="" type="checkbox"/>	...
Node 5	Point of Compliance		None	Copy	<input type="checkbox"/>	...
Node 6	Node 6		None	Copy	<input type="checkbox"/>	...
Node 7	Node 7		None	Copy	<input type="checkbox"/>	...
Node 8	Node 8		None	Copy	<input type="checkbox"/>	...
Node 9	Node 9		None	Copy	<input type="checkbox"/>	...
Node 10	Node 10		None	Copy	<input type="checkbox"/>	...

Watershed Schematic

Project Location Land Use **Network** Runoff/Optimize Graphics Water Quality Tools

3:45 PM

Figure 8.5– Postdevelopment Network Input Screen for Subbasin/Node Connection Example

## 9 Link Definitions and Parameters

Nodes are used to collect runoff from the tributary area for a given subbasin and from the nodes of upstream subbasins. Links are used to connect one node to another and may be defined as one of the following:

1. Copy – Copies discharge from upstream node and adds it to discharge at the downstream node without routing,
2. Structure – Includes detention and infiltration ponds, and sand filters,
3. Channel – Performs routing in open channels without infiltration,
4. Infiltration Trench – Performs routing through infiltration trenches and can be used to simulate open channel routing with infiltration,
5. Rating Table – User defined stage storage discharge table,
6. Flow Splitter – Splits a fraction of the discharge from one node to another.

Information for each type of Link is discussed in the following Sections.

### 9.1 Copy Link

The copy link copies timeseries from the upstream node and adds it to the downstream node. Hydrographs are transferred to the downstream node without attenuation or lagging. Thus, there are no parameters to input for a Copy link. The copy link is appropriate for small watersheds where there is little attenuation of the flood hydrograph from one node to the next. If the conveyance channel is long with large overbank storage, then the link should be defined as an open channel. As a general rule, channel routing may be neglected for watersheds smaller than about  $\frac{1}{2}$  square mile (320 acres) and the link may be defined using the copy routine.

### 9.2 Structure Link

Structure links are used to define stormwater ponds, infiltration ponds, and sand filters. Pond optimization information for post-development condition ponds is also input on the structure link input screens.

A variety of hydraulic devices can be included in the design of stormwater treatment facilities. Devices attached to the riser structure include; circular orifices, circular orifices under backwater influence, rectangular orifices, rectangular weirs, V-notch weirs, and proportional weirs. In addition, the riser structure can also be defined with an open top to function as an overflow weir, or the top may be capped. Any combination of up to six devices plus the riser structure and a sand filter can be included for each structure. A trapezoidal broad crested weir may also be specified to function as an emergency overflow.

The following sections describe the input for Structure Links.

### 9.2.1 Pond/Vault Geometry Input

Two options are available for specifying pond or vault geometry. The first assumes a prismatic geometry with pond length, width, depth, and side slopes as shown in Figure 9.1.

**Structure Input Data - New Pond**

**Pond/Vault Geometry** | Outlet Structure(s) | Optimization Data | Sand Filter Data

Structure Name:

☒ Use Prismatic Pond Geometry ☐ Use Elevation Volume Table

Max Pond Elevation (ft):

**Prismatic Pond/Vault Geometry**

	Z1	Z2	Z3	Z4
Side Slopes (ZH:1V)	<input type="text" value="3.00"/>	<input type="text" value="3.00"/>	<input type="text" value="3.00"/>	<input type="text" value="3.00"/>
Pond Bottom Length, L (ft)	<input type="text" value="368.38"/>			
Pond Bottom Width, W (ft)	<input type="text" value="184.18"/>			
Pond Floor or Top of Dead Storage Elevation (ft)	<input type="text" value="100.00"/>			

**Pond Bottom Area:** 67848. sq ft

**Pond Volume At:** Riser Crest Elevation: 218769. cu ft, (5.022 ac-ft)  
Maximum Pond Elevation: 258260. cu ft, (5.929 ac-ft)

**Pond Infiltration Data**

Soil Hyd Conductivity (in/hr):  ☒ Low Bio-Fouling Potential

Depth to Water Table (ft):  ☒ Average or Better Maintenance

☐ User Defined Elevation Volume Table

**Plan View**

**Elevation View**

Max Pond Elev.

Pond Floor or Top of Dead Storage

Riser Structure

i (in/hr)

1/Z

W

L

Z1

Z2

Z3

Z4

Ok Cancel

Figure 9.1 – Hydraulic Structures Input Screen

where:

$L$  – is the pond length in feet,

$W$  – is the pond width in feet,

$Z1, Z2, Z3, Z4$  - are side slopes for each side of the pond where  $Z$  is the number of feet in the horizontal plane for every foot of rise,

**Pond Floor Elevation** – Represents the bottom of the *live* pond storage. Live storage is defined as the storage used to detain stormwater runoff and eventually flows through the outlet structure. Dead storage is retained in the pond below the elevation of the outlet structure. The pond floor elevation should be input if the pond is not a combined wet pond. If the pond is a combined wet pond, then enter the elevation of the top of the dead storage, i.e. the elevation where water begins to discharge from the pond

*Riser Crest Elevation* – The elevation at which water begins to flow into the overflow riser. The maximum flood recurrence interval detained by the pond generally corresponds with this elevation (or slightly above this elevation). For example, the Ecology flow duration standard requires control of the flow duration between ½ of the 2-year and the 50-year recurrence interval. Water will begin to spill into the riser structure near the 50-year recurrence interval. It is acceptable for water to spill into the riser structure for floods smaller than the 50-year provided that the flow duration standard is met.

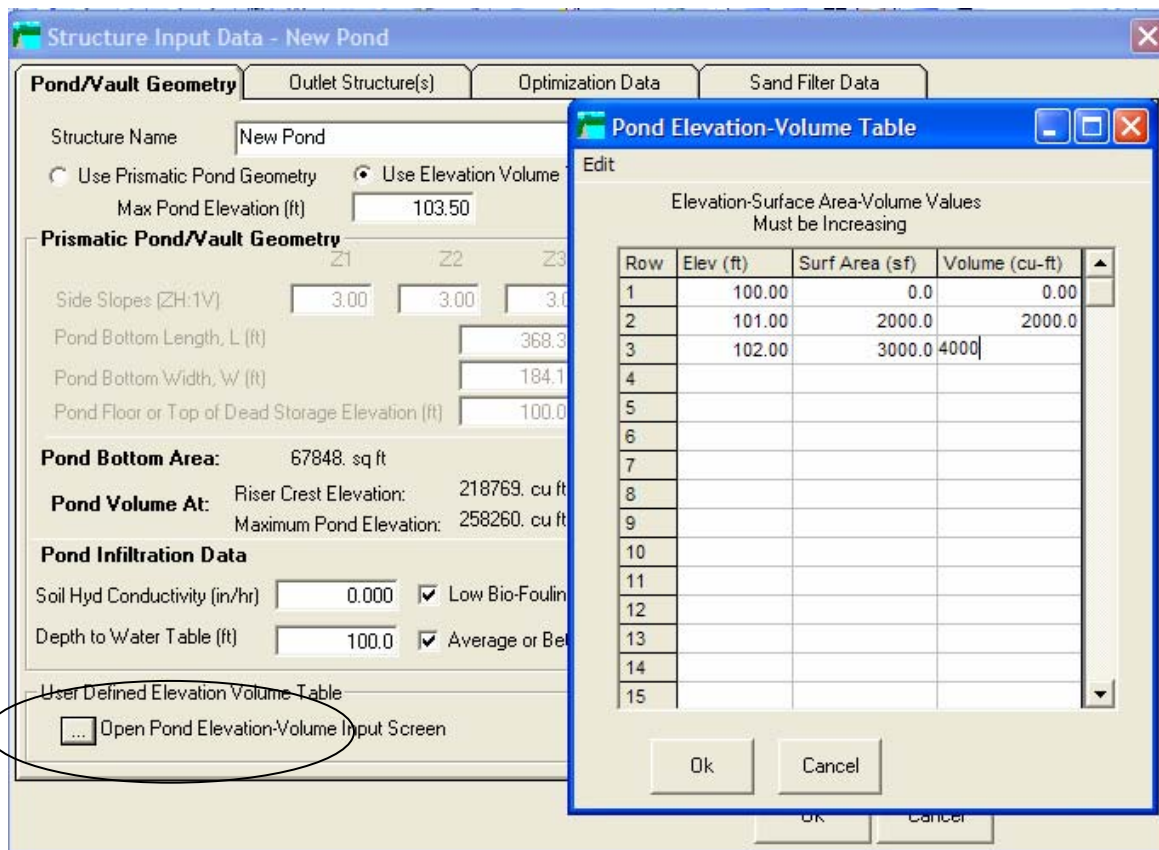
*Max Pond Elevation* – Is the maximum elevation used in pond routing calculations and typically extends above the riser crest elevation a sufficient distance to accommodate large floods or to allow for flood passage if one or more of the lower level outlets become blocked. The required maximum pond elevation depends on the design standards of the local jurisdiction.

The automatic pond sizing routine (optimizer) in MGSFlood determines the riser diameter and maximum pond elevation so that the 100-year peak inflow will pass through the riser structure assuming the lower level outlets are blocked. The user is advised to check the maximum pond elevation returned by the optimizer with the design standards of the local jurisdiction including any freeboard requirements.

If a vault is to be analyzed, then side slopes (*Z1, Z2, Z3, Z4*) of zero are input denoting vertical sides. The pond volume for elevations ranging from the floor to one foot above the maximum pond elevation is computed according to this geometry.

The second method for specifying pond geometry is with a user defined elevation-volume table as shown in Figure 9.2. This is useful for specifying the geometry of irregularly shaped ponds. The elevation-volume relationship can be computed using a spreadsheet program and pasted into the form using the Windows Clipboard utility.

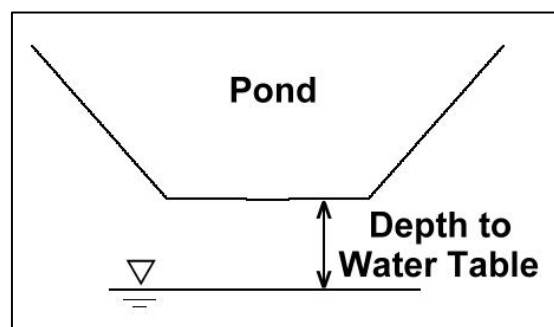
Note: Precipitation falling on the surface of the detention pond is not automatically computed by MGSFlood. This approach was taken to allow use of both ponds and vaults. The difference being ponds are open to collection of precipitation, and vaults are closed to precipitation input. To include precipitation on the pond surface in the computations, the surface area of the pond must be included under the land use for the subbasin where the pond resides. This can be accomplished by adding impervious surface equal to the maximum pond surface area under the *Subbasin Definitions* window for the sub-basin where the pond resides. A simple approach to get an initial estimate of the pond surface area would be to run the *Quick Optimization routine* after the tributary subbasins have been defined.



**Figure 9.2 – Hydraulic Structures Input Screen, User Defined Elevation/Volume Input**

### 9.2.2 Pond Infiltration

MGSFlood simulates infiltration using an approach developed by Massmann<sup>30</sup> based on field observations of infiltration ponds in western Washington. This infiltration approach accounts for the side slope geometry of the pond, pond aspect (length to width ratio), the proximity of the pond to the regional groundwater table, and the potential for soil clogging and fouling. Inputs include; Soil Hydraulic Conductivity (inches/hour), Depth to the Regional Water Table (ft) (Figure 9.3), whether bio-fouling potential is low, and whether average or better maintenance is performed. Infiltrated moisture is lost from the system and does not contribute to the discharge rate through the riser or orifices.



**Figure 9.3 – Infiltration Pond Depth to Water Table  
(Accounts for Groundwater Mounding Beneath Pond)**

Soil Hydraulic Conductivity (in/hr) – Is the saturated hydraulic conductivity of the soil beneath the pond in inches per hour according to Darcy's Equation. It can be estimated using regression equations that use grain size distribution as input (Massmann<sup>30</sup>) or from literature (e.g. Freeze and Cherry<sup>31</sup>, Fetter<sup>32</sup>).

Depth to Regional Groundwater Table (ft) – Represents the depth from the bottom of the pond to the regional groundwater table or the first low-permeability layer. For shallow groundwater sites, groundwater mounding reduces the hydraulic gradient and the infiltration rate is significantly less than the saturated hydraulic conductivity. For deep groundwater sites where the effects of mounding will be small, the gradient will not typically be reduced by infiltration from the facility. Increasing the depth to groundwater greater than 100 feet ceases to have an influence on pond infiltration according to this approach.

Bio-fouling Potential – Bio-fouling occurs from organic material blanketing the soil surface and reducing the infiltration rate. Bio-fouling is more likely to occur if the pond is located beneath trees and other vegetation or in shaded locations.

Maintenance – Siltation is more likely to occur if there is not sufficient pre-treatment of the storm water or in locations where the drainage basin is prone to erosion because of recent land disturbances or steep slopes. The user should consider the potential for siltation of the infiltration pond and the maintenance program when determining the effects of maintenance on pond infiltration performance.

### 9.2.3 Outlet Structures

The *Outlet Structures* Tab defines type, size, and elevation of pond outlets (Figure 9.4). Up to eight outlet devices consisting of any combination of the following can be defined:

- Circular orifice with or without tailwater,
- Rectangular orifice or slot,
- V-notch sharp crested weir,
- Rectangular sharp crested weir,
- Proportional sharp crested weir
- Trapezoidal broad crested weir

If an orifice subjected to tailwater is selected, then an elevation-discharge rating table must be entered by clicking the *Tailwater* button (Figure 9.5). A minimum of four elevation-discharge pairs must be entered, and discharges must be entered in an increasing order of magnitude. See Section I, Chapter 8 of the users manual for more information regarding the geometry and hydraulic equations governing each structure, and guidance for backwater conditions.

**Structure Input Data - New Pond**

Pond/Vault Geometry   **Outlet Structure(s)**   Optimization Data   Sand Filter Data

**Control Orifice/Weir Structures**

Enable	Structure Type	Control El. (ft)	Diameter (in)	Orientation	Elbow
<input checked="" type="checkbox"/>	Circular Orifice	100.00	1.022	<input checked="" type="radio"/> Horizontal <input type="radio"/> Vertical	<input type="radio"/> Yes <input checked="" type="radio"/> No
<input checked="" type="checkbox"/>	Rectangular Orifice	102.20	1.388	Height (in): 9.65	<input type="radio"/> Horizontal <input checked="" type="radio"/> Vertical
<input type="checkbox"/>	Circular Orifice			<input checked="" type="radio"/> Horizontal <input type="radio"/> Vertical	<input checked="" type="radio"/> Yes <input type="radio"/> No
<input type="checkbox"/>	Circular Orifice			<input checked="" type="radio"/> Horizontal <input type="radio"/> Vertical	<input checked="" type="radio"/> Yes <input type="radio"/> No
<input type="checkbox"/>	Circular Orifice			<input checked="" type="radio"/> Horizontal <input type="radio"/> Vertical	<input checked="" type="radio"/> Yes <input type="radio"/> No
<input type="checkbox"/>	Circular Orifice			<input checked="" type="radio"/> Horizontal <input type="radio"/> Vertical	<input checked="" type="radio"/> Yes <input type="radio"/> No

**Riser Structure**

Structure Type	Crest El. (ft)	Diameter (in)	Common L (ft)	Riser Top Open
Circular Overflow Riser	103.00	18.00	0.12	<input checked="" type="radio"/> Yes <input type="radio"/> No

Ok   Cancel

Figure 9.4 – Hydraulic Structures Input Screen

Low Orifice Elevation-Discharge Values

Edit

Elevation Discharge Values for Low Orifice  
Values Must be Increasing

Row	Elev (ft)	Disch (cfs)
1		
2		
3		
4		
5		
6		
7		
8		
9		
10		
11		

Ok Cancel

Figure 9.5 – Input Screen for Specifying Tailwater Conditions for Low Orifice

#### 9.2.4 Riser Structure

The pond riser structure is defined at the bottom of the *Outlet Structures* tab (Figure 9.4). A single riser structure is defined for each pond and can be either circular or rectangular in cross section. If the *Riser Top Open* option button is selected, then the riser functions as an overflow structure. The *Common Length* field defines the sum of any outlet structures that intersect the riser crest. Specifying the common length ensures that the discharge from the hydraulic structure is not double counted when flow passes over the riser crest (See Section I, Chapter 8 for more details).

#### 9.2.5 Automatic Pond and Outlet Works Sizing Routine/Optimization

The pond sizing optimization routine automatically determines the size of the pond, and size and elevation of the outlet works needed to meet the Washington State Department of Ecology Flow Duration Standard<sup>9</sup>.

Designing stormwater ponds to this standard is a laborious, iterative process when performed manually. In addition, because of the number of variables involved in designing a pond to the flow duration standard, it is difficult to find a pond configuration that minimizes the pond volume and meets the duration standard using manual trial and error. The automatic pond sizing routine seeks to determine a minimum pond size that meets the flow duration standard.

The automatic pond sizing optimization routine will determine the pond size and outlet configuration for two pond types; a detention pond with minor infiltration and an infiltration pond. The characteristics of these two pond types are listed in Table 9.1

Table 9.1 – Characteristics of Detention and Infiltration Ponds Sized using Optimization Routine		
Characteristic	Detention Pond	Infiltration Pond
Pond Configuration	Riser Structure with Low Level Circular Orifice and Vertical Rectangular Upper Orifice	Overflow Riser Only
Valid Infiltration Rates*	0.00 – 0.10 inches/hour	0.05-50 inches/hour
Optimization Levels	Quick or Full	Quick Only

\* Note: Infiltration occurs through the pond bottom only, not including the side slopes.

The pond sizing optimization routine uses general input about the pond geometry including;

- Pond length to width ratio,
- Pond side slopes (Z1, Z2, Z3, Z4),
- Pond floor elevation,
- Riser crest elevation,
- Hydraulic Conductivity (used to simulate infiltration),
- Depth to Water Table (used to simulate infiltration),
- Bio-fouling Potential and Maintenance Level (used to simulate infiltration).

These variables are entered on the Optimization tab (Figure 9.6). The Optimize Box is checked for the link to be optimized on the *Network* tab. The link will automatically be optimized when routing is performed on the *Runoff/Optimize* tab.

Figure 9.6 – Automatic Pond Design and Optimization Input Screen

The optimization routine is currently configured to handle modest amounts of infiltration for detention ponds, with infiltration rates less than 0.10 in/hr. If a larger infiltration rates are required, it is recommended that the quick optimization routine be used to obtain a rough starting point and then proceed with the pond design using the manual adjustment.

To manually edit the pond configuration determined by the *Optimization* routine, reopen the *Hydraulic Structures* input screen from the *Network* tab and make changes to the pond or outlet works geometry returned by the program. Uncheck the *Optimize Structure* box on the *Runoff/Optimize* tab and click the *Route* button to route flows through the network. Guidelines for manually adjusting the outlet works and pond geometry to achieve compliance with the flow duration standard are listed in Chapter 14 of Part I.

### **Optimization Level**

Two levels of optimization are available for detention pond sizing; Quick Optimization and Full Optimization. *Quick Optimization* determines a “ballpark” solution in a relatively short time (usually less than one minute). *Full Optimization* does an exhaustive search of potential solutions in searching for a configuration for the minimum pond size required to meet the flow duration standard. The full optimization routine usually converges on a solution in less than ten minutes (depending on the speed and memory of the computer).

For infiltration ponds, only *Quick Optimization* is available. Infiltration pond optimization is much less computationally demanding than detention pond optimization and the Quick Optimization routine typically produces an optimal pond design in a short period of time.

### **9.2.6 Running the Pond Optimization Routine**

Optimization of selected ponds or infiltration trenches on the postdevelopment network will be performed when routing is performed by clicking the *Route* button on the *Runoff/Optimize* tab. The *Optimize Structure Indicated on Network Tab* check box must be selected for optimization to occur. Only one structure may be optimized per simulation run. To optimize multiple structures, start with the furthest upstream structure and optimize each structure working downstream.

When the Pond Optimization routine is executed, a second window opens that displays progress messages from the routine (Figure 9.7). If the *Full Optimization* option is checked when sizing a detention pond, a matrix will be displayed on the screen and filled with symbols indicating the progress of the routine. When the routine is finished, the pond size and outlet information is placed on *Structure Input* screen for the link being optimized replacing any previously entered information. The program then automatically computes and plots the pond performance duration statistics. If the resulting pond does not meet all of the duration design criteria, then manual edits to the pond design must be made. The procedure for making manual edits to a pond returned by the optimization is described in the next section.

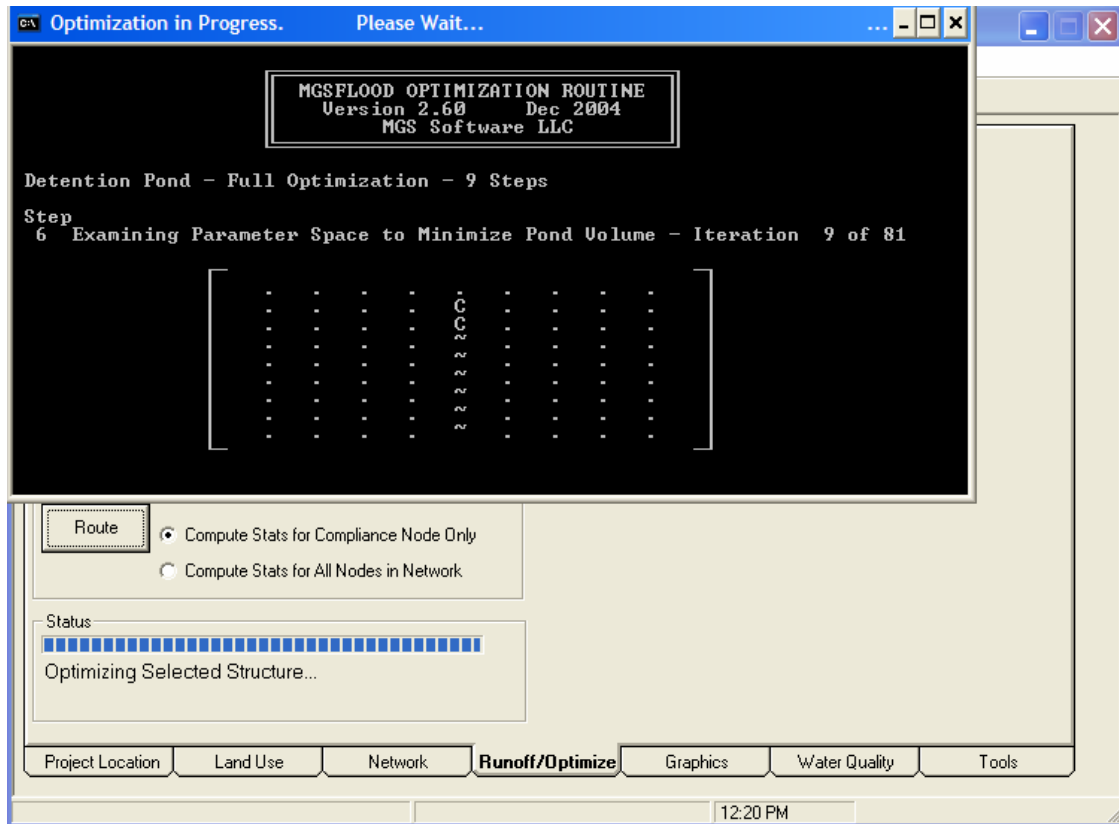


Figure 9.7 – MGSFlood Pond Optimization Status Screen

### 9.2.7 Sand Filter

A sand filter functions much like an infiltration pond except that instead of infiltrating into native soils, stormwater filters through a constructed sand bed with an underdrain system to remove pollutants. The underdrain system is assumed connected to the discharge conduit from the pond and flows from the sand filter are added to the total discharge from the pond. The program treats the sand filter as an additional structure associated with the stormwater pond. The filter surface area is used by the program to determine the rate of water infiltrated through the filter. The pond length and width entered on the Pond/Vault geometry are used to establish the pond storage volume. The pond bottom area may be larger than the sand filter area to allow placement of the sand filter in just a portion of the pond bottom.

To include a sand filter, check the *Include Sand Filter* box on the *Sand Filter Data* tab (Figure 9.7). The elevation of the top of the filter, filter surface area, thickness, and permeability are entered on the input screen (See Section I for more information regarding these parameters).

**Structure Input Data - New Pond**

Pond/Vault Geometry    Outlet Structure(s)    Optimization Data    **Sand Filter Data**

☒ Include Sand Filter

Sand Filter Data

Elevation of Top of Filter (ft)	100.00
Filter Surface Area (Square Feet)	5000.00
Filter Thickness (ft)	3.00
Permeability (inches/hour)	1.500

**Notes:**

1. Filtered water is assumed to be collected by an under drain system that discharges to the downstream node.
2. Infiltration through the pond floor is not allowed when a sand filter is present (Include Sand Filter Box Checked Above).
3. Any infiltration information entered on previous tabs for this structure will be ignored if Sand Filter Box is Checked.
4. Sand filter performance information (% of runoff filtered) is recorded in the project report following routing.
5. Sand filters are not used by the pond optimizer. The Sand Filter checkbox will be unchecked following optimization.

Ok    Cancel

**Figure 9.7 – Sand Filter Input Screen**

### 9.3 Channel Routing

Channel routing is performed using a Modified Puls routing routine developed by the US Army Corps of Engineers for the HEC-1<sup>25</sup> flood hydrograph package. The user inputs the left overbank, main channel and right overbank channel cross sectional geometry, roughness, slope, and channel length (Figure 9.8). The program develops an elevation-volume-discharge rating table assuming normal depth at each discharge level and computes discharge according to the Manning Equation<sup>4</sup>. This rating table is then utilized by the Modified Puls routing routine to route flows from the upstream to the downstream node. To simulate infiltration through the channel bottom, define the link as an infiltration trench and set the depth of gravel to zero (See Section 9.4).

### Post-Developed Network: New

☒ View Graph of Input Data  
☐ View Data Input Definitions

Channel Name:

LEFT OVERBANK		MAIN CHANNEL		RIGHT OVERBANK	
Upper Sideslope (Z)	<input type="text" value="0.50"/>	Lower Sideslope Left (Z)	<input type="text" value="0.50"/>	Upper Sideslope (Z)	<input type="text" value="0.50"/>
Upper Width, W (ft)	<input type="text" value="3.00"/>	Lower Width Left W (ft)	<input type="text" value="3.00"/>	Upper Width W (ft)	<input type="text" value="3.00"/>
Middle Sideslope (Z)	<input type="text" value="10.00"/>	Lower Sideslope Right (Z)	<input type="text" value="0.50"/>	Middle Sideslope (Z)	<input type="text" value="10.00"/>
Middle Width, W (ft)	<input type="text" value="10.00"/>	Lower Width Right W (ft)	<input type="text" value="3.00"/>	Middle Width W (ft)	<input type="text" value="10.00"/>
Mannings n Roughness	<input type="text" value="0.040"/>	Mannings n Roughness	<input type="text" value="0.024"/>	Mannings n Roughness	<input type="text" value="0.040"/>
		Base Width W (ft)	<input type="text" value="10.00"/>		
		Base Elevation (ft)	<input type="text" value="100.00"/>		
		Channel Bed Slope (ft/ft)	<input type="text" value="0.0200"/>		
		Channel Length (ft)	<input type="text" value="1000.0"/>		

Figure 9.8 – User Input for defining Open Channel Routing

## 9.4 Infiltration Trench

MGSFlood simulates infiltration using an approach developed by Massmann<sup>30</sup> based on field observations of infiltration ponds in western Washington. This approach accounts for the side slope geometry of the structure, the aspect (length to width ratio), the proximity to the regional groundwater table, and the potential for soil clogging and fouling. Inputs include; Soil Hydraulic Conductivity (inches/hour), Depth to the Regional Water Table (ft), whether bio-fouling potential is low, and whether average or better maintenance is performed. Infiltrated moisture is lost from the system and does not contribute to the discharge rate at the downstream node.

The program routes flow for two types of infiltration trenches as shown in Figures 9.9 and 9.10; a trench located on the embankment side slope, or an infiltration trench located at the base of the embankment.

**Channel Routing Definition (Post): New**

**Trench Geometry** | Optimization Data

**Infiltration Trench on Embankment Slope**

Diagram labels: Road, Embankment Slope, Gravel Filled Trench, Width, Trench Depth, Depth to Water Table

Structure Name: New

Trench Bottom Elev at Downstream End (ft): 100.00

Trench Length (ft): 300.0

Trench Depth (ft): 3.00

Trench Width (ft): 35.00

Rock Fill Porosity % (Vol Voids/Tot Vol): 50.0

Saturated Hydraulic Conductivity (in/hr): 6.00

Depth to Water Table Beneath Trench (ft): 100.0

☒ Average or Better Maintenance

☒ Low Bio-Fouling Potential

☒ Trench Located on Embankment Sideslope

☐ Trench Located Beneath Ditch

Ok Cancel

Figure 9.9 – Infiltration Trench Located on Embankment Slope Option

**Channel Routing Definition (Post): New**

**Trench Geometry**      Optimization Data

**Standard Infiltration Trench**  
(View Looking Downstream)

Structure Name:

Trench Bottom Elev at Downstream End (ft):

Trench Length (ft):

Trench Depth (ft):

Trench Width (ft):

Rock Fill Porosity % (Vol Voids/Tot Vol):

Saturated Hydraulic Conductivity (in/hr):

Depth to Water Table Beneath Trench (ft):

☒ Average or Better Maintenance

☒ Low Bio-Fouling Potential

☐ Trench Located on Embankment Sideslope

☒ Trench Located Beneath Ditch

Trench Sideslope Left (ZH:1V):       Ditch Bedslope (ft/ft):

Trench Sideslope Right (ZH:1V):       Ditch Mannings n Roughness:

Ok      Cancel

**Figure 9.10 – Standard Infiltration Trench Option**

Soil Hydraulic Conductivity (in/hr) – Is the saturated hydraulic conductivity of the soil beneath the infiltration trench in inches per hour according to Darcy's Equation. It can be estimated using regression equations that use grain size distribution as input (Massmann<sup>30</sup>) or from literature (e.g. Freeze and Cherry<sup>31</sup>, Fetter<sup>32</sup>).

Depth to Regional Groundwater Table (ft) – Represents the depth from the bottom of the trench to the regional groundwater table or the first low-permeability layer. For shallow groundwater sites, groundwater mounding reduces the hydraulic gradient and the infiltration rate is significantly less than the saturated hydraulic conductivity. For deep groundwater sites where the effects of mounding will be small, the gradient will not typically be reduced by infiltration from the facility. Increasing the depth to groundwater greater than 100 feet ceases to have an influence on pond infiltration according to this approach.

Bio-fouling Potential – Bio-fouling occurs from organic material blanketing the soil surface and reducing the infiltration rate. Bio-fouling is more likely to occur if the trench is located beneath trees and other vegetation or in shaded locations.

Maintenance – Siltation is more likely to occur if there is not sufficient pre-treatment of the storm water or in locations where the drainage basin is prone to erosion because of recent land disturbances or steep slopes. The user should

consider the potential for siltation and the level of maintenance when determining the effects of maintenance on pond infiltration performance.

## 9.5 Infiltration Trench Located on Embankment Slope

A trench is constructed along the roadway embankment and filled with gravel (Figure 9.9). Runoff from the roadway is directed to the gravel trench where it percolates through the gravel and infiltrates through the trench bottom. When the runoff rate exceeds the infiltration capacity, the gravel saturates from the bottom up with the voids in the gravel providing runoff storage, similar to a detention pond. If the storm is sufficiently large, the saturation will reach the ground surface and runoff from the road will pass over the gravel surface and continue down the embankment. Runoff not infiltrated in the trench is passed to the downstream node without routing.

It should be noted that the saturated hydraulic conductivity of the embankment fill will likely be different from the native material beneath the fill. The hydraulic conductivity estimates of the different layers can be combined using the harmonic mean (Massmann<sup>30</sup>):

$$K_{Equiv} = \frac{d}{\sum \frac{d_i}{K_i}} \quad 9.1$$

Where:  $K_{Equiv}$  is the equivalent hydraulic conductivity,  
 $d$  is the depth of the soil column above the regional groundwater table or limiting permeability layer,  
 $d_i$  is the thickness of layer  $i$ ,  
 $K_i$  is the hydraulic conductivity of layer  $i$

Note that the saturated hydraulic conductivity of the gravel in the trench is not included in Equation 9.1. For sites with very deep groundwater tables (>100 feet), it is recommended that the total depth of the soil column in Equation 10.1 be limited to 20 times the trench depth.

## 9.6 Standard Infiltration Trench

The standard infiltration trench would be constructed at the base of the roadway embankment and would receive runoff from the adjacent roadway or from an upstream ditch. Runoff from the roadway is directed to the gravel trench where it percolates through the gravel and infiltrates through the trench bottom. When the runoff rate exceeds the infiltration capacity of the soil, the gravel saturates from the bottom up with the voids in the gravel providing runoff storage, similar to a detention pond. If the storm is sufficiently large, the saturation will reach the ground surface and runoff will occur down the ditch along the gravel surface. The program routes flow along the gravel surface to the downstream node according to the Manning Equation<sup>4</sup>.

The infiltration trench routine may also be used to simulate a natural stream channel with infiltration through the channel bottom. The geometry of the channel is defined as a trapezoidal section and depth of gravel is input as zero.

## 9.7 Automatic Infiltration Trench Sizing Routine

The automatic pond sizing optimization routine in the MGSFlood will automatically determine the size of infiltration trench required to meet the goals of the Ecology flow duration standard. The optimization routine uses general input about the infiltration trench and solves for the trench bottom width. The input supplied by the user includes:

- ❖ The type of infiltration trench to be sized (Embankment Slope or Standard),
- ❖ The trench bottom elevation at the downstream end,
- ❖ Trench depth,
- ❖ Trench length,
- ❖ Rock fill porosity,
- ❖ Depth to water table,
- ❖ Saturated hydraulic conductivity of soil beneath trench.

The optimization routine uses the information listed above to establish the geometric relationships for the trench configuration. The program establishes a parameter space of possible solutions by varying the bottom width. The program then routes the developed runoff timeseries through the trench and seeks to find a solution that provides the minimum bottom width to meet the duration design standard.

These variables are entered on the Optimization tab (Figure 9.11). The Optimize Box is checked for the link to be optimized on the *Network* tab. The link will automatically be optimized when routing is performed on the *Runoff/Optimize* tab.

**Channel Routing Definition (Post): New**

**Optimization Data**

**Type of Trench**

- ☐ Trench Located on Embankment Sideslope
- ☒ Trench Located Beneath Ditch

**Initial Structure Geometry for Optimization**

Trench Bottom Elev at Downstream End (ft)	100.00	<input checked="" type="checkbox"/> Low Bio-Fouling Potential
Trench Length (ft)	300.0	<input checked="" type="checkbox"/> Average or Better Maintenance
Trench Depth (ft)	3.00	
Rock Fill Porosity % (Vol Voids/Tot Vol)	50.0	
Saturated Hydraulic Conductivity (in/hr)	6.00	
Depth to Water Table Beneath Trench (ft)	100.0	

Trench Sideslope Left (ZH:1V)	3.00	Ditch Bedslope (ft/ft)	0.020
Trench Sideslope Right (ZH:1V)	3.00	Ditch Mannings n Roughness	0.024

Ok Cancel

Figure 9.11 – Optimization Input Screen for Infiltration Trenches

## 9.8 User Defined Rating Table

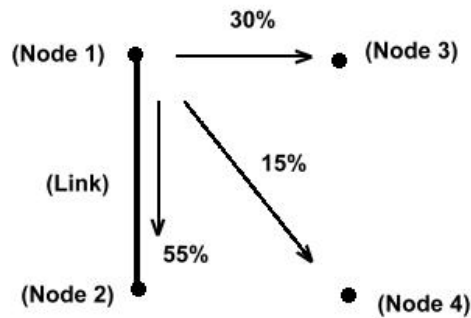
Structure hydraulics may be specified using a user-defined stage-surface area-volume-discharge rating table (Figure 9.12). The pond storage (acre-feet), surface area (acres), discharge (cfs), and infiltration (inches per hour) are computed by the user and entered in the table. Information may be copied from an external spreadsheet program and pasted into the input table using the Windows Clipboard utility.

Row	Elev (ft)	Area (ac)	Vol (ac-ft)	Disch (cfs)	Infiltr (cfs)
1	0.00	0.000	0.0000	0.000	0.0000
2	0.50	1.180	0.2900	0.000	0.0660
3	1.00	1.190	0.8900	0.000	0.0750
4	1.50	1.210	1.4900	0.000	0.0830
5	2.00	1.240	2.1000	0.000	0.0930
6	2.50	1.270	2.7300	0.000	0.1020
7	3.00	1.300	3.3700	0.000	0.1130
8	3.50	1.340	4.0300	0.000	0.1250
9	4.00	1.380	4.7100	0.000	0.1370
10	4.50	1.430	5.4100	0.000	0.1500
11	5.41	1.520	6.7500	0.000	0.1770
12	5.50	1.530	6.8900	0.810	0.1800
13	6.00	1.590	7.6700	2.080	0.1970
14	6.50	1.660	8.4800	2.130	0.2150
15	7.00	1.730	9.3300	2.670	0.2340
16	7.50	1.800	10.2100	3.140	0.2550
17	8.00	1.880	11.1300	3.550	0.2780

Figure 9.12 – Input Screen for User Defined Rating Table

## 9.9 Flow Splitter

Flow splitter structures allow runoff collected at one node to be split between two or more downstream nodes. Input consists of a table that specifies the fractional amount of discharge split among the downstream nodes. Figure 9.13 shows an example flow splitter defined for a link connecting Node 1 with Node 2. The flow splitter is configured with 30% of the runoff split to Node 3, 15% to Node 4, and the remaining 55% to Node 2



Upstream Discharge at Node 1 (cfs)	Fraction to Node 3	Fraction to Node 4	Remaining Fraction to Downstream Node 2
0	0.30	0.15	0.55
10	0.30	0.15	0.55
100	0.30	0.15	0.55

Figure 9.13 – Flow Splitter Example

## 10 Runoff/Optimize Tab

After inputting land use, connecting subbasins to nodes, and defining links connecting nodes, runoff and routing computations are performed from the *Runoff/Optimize* tab (Figure 10.1). MGSFlood computes runoff using the impervious (IMPLND) and pervious (PERLND) land segment subroutines from the HSPF model. Hourly precipitation and evaporation are read from the MGSRegion.mdb file, runoff is computed for predevelopment and postdevelopment conditions. Routing through the predeveloped and postdeveloped networks is then performed followed by calculation of statistics with results displayed on the *Graphics* tab.

### 10.1 Specify Time Period for which Runoff is to be Computed

Runoff computations are performed on a *water year* basis, that is, they begin on October 1 and end on September 30. This is done because the soils are typically driest at the beginning of fall and a single set of antecedent conditions can be used for all regions of western Washington. The user can define a time period shorter than the full record for the runoff computations, although the full period of record should be used in facility design to provide the most accurate streamflow computations.

### 10.2 Optimization Option

Optimization of selected ponds or infiltration trenches on the postdevelopment network will be performed if the *Optimize Structure Indicated on Network Tab* check box is selected. Only one structure may be optimized per simulation run. To optimize multiple structures, start with the furthest upstream structure and optimize each structure working downstream.

### 10.3 Compute Statistics Option Buttons

Two options are available for calculating statistics in the project. Statistics may be computed for the compliance nodes only (lower output level) or all nodes in the project (higher output level). Water surface elevation statistics are also computed for links connected to the compliance node (lower output level) or all links if the higher output level is selected.

### 10.4 Compute Runoff Run Button

Clicking the Run button causes runoff to be computed for the period selected and runoff to be saved in the direct access file for the nodes selected. The program then reads the runoff stored in this file for all future pond sizing calculations for the project. If the land use is subsequently changed, the runoff must be recomputed.

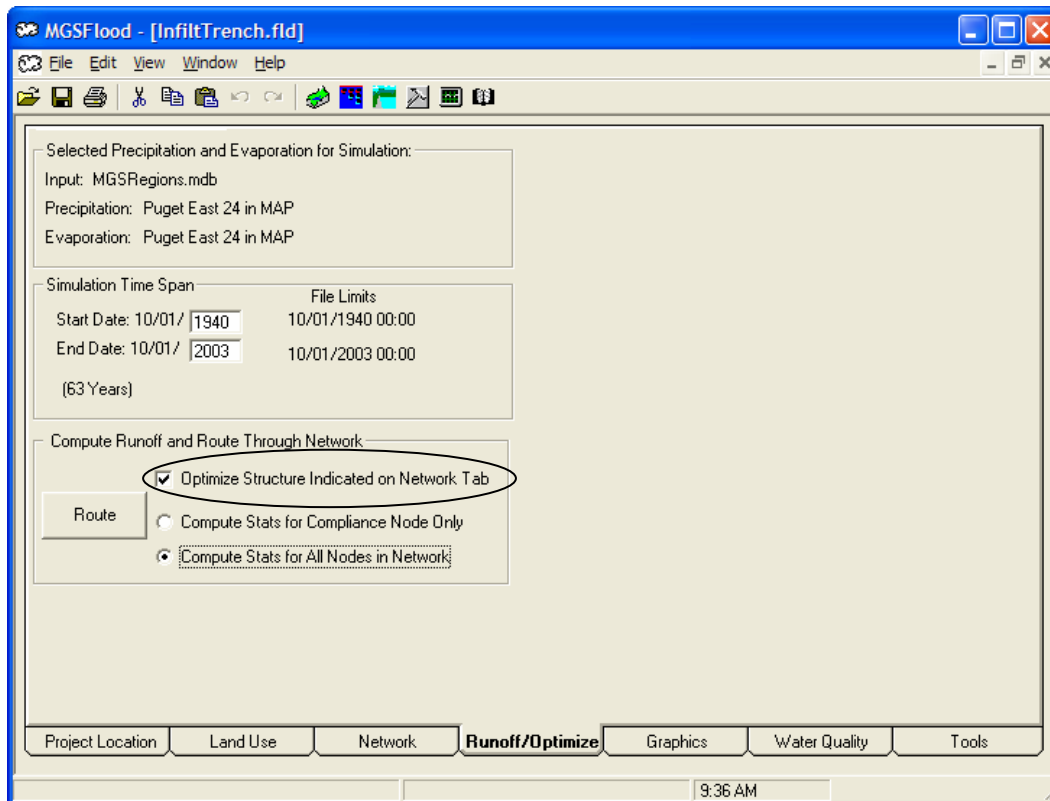


Figure 10.1 – Runoff/Optimize Input Tab

## 10.5 Manual Editing of Pond Configuration Obtained from the Optimization Routine

To manually edit the configuration of a pond or infiltration trench determined by the *Optimization* routine, reopen the *Structure Input* screen using the *Link Definition* button from the *Network* tab and make changes to the pond or trench geometry returned by the program. On the *Runoff/Optimize* tab, uncheck the *Optimize Structure Indicated on the Network Tab* then click the *Route* button. This will route flows without rerunning the optimization routine. Guidelines for manually adjusting the outlet works and pond geometry to achieve compliance with the flow duration standard are listed in Chapter 9 of Part I.

## 11 Graphs Tab

The *Graphs* tab is used for plotting runoff statistics for selected nodes, plotting the performance of a stormwater treatment facilities or plotting hydrographs from selected nodes.

The type of graph to be plotted is determined by the *Plot Type* option buttons;

- ❖ Flood Frequency,
- ❖ Flow Duration,
- ❖ Water Surface Elevation (WSEL) Frequency in ponds,
- ❖ Hydrographs.

The node or links to be plotted are selected using the drop down list boxes for the predeveloped and postdeveloped condition (Figure 11.1).

### 11.1 Flood Frequency Statistics Graphs

Flood frequency statistics are plotted by selecting the *Flood Frequency* option button and clicking the *Draw* button. Each time the draw button is clicked, the graph on the screen and the jpeg file on disk are each updated.

### 11.2 Water Surface Elevation Statistics

Water surface elevation statistics are available for any link defined as a pond. Flood frequency statistics are plotted by selecting the *Flood Frequency* option button and clicking the *Draw* button. The pond bottom and riser crest elevations are noted on the graphs. Each time the draw button is clicked, the graph on the screen and the jpeg file on disk are each updated.

### 11.3 Flow Duration Statistics Graphs

Flow duration statistics are plotted by selecting the *Flow Duration* option button and clicking the *Draw* button. Each time the draw button is clicked, the graph on the screen is updated and the graph is stored onto disk as a jpeg file. For the compliance nodes, the graph includes annotations noting the predeveloped  $\frac{1}{2}$  of the 2-year, 2-year and the 50-year flows, the exceedance probability corresponding to these flows, whether the Department of Ecology Flow Duration criteria<sup>9</sup> have been met, and the maximum water surface elevation occurring in the pond during the simulation (Figure 11.1).

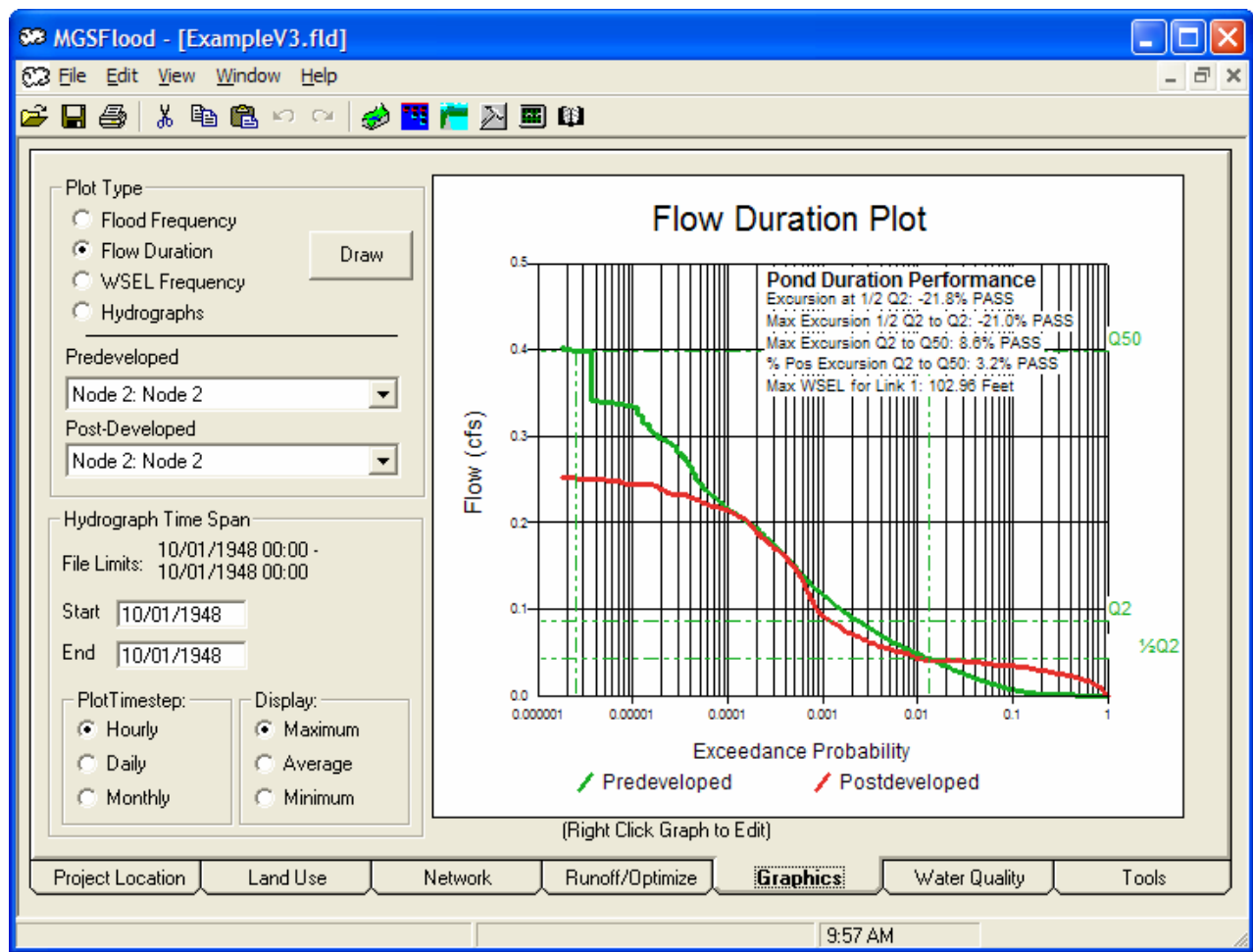


Figure 11.1 – Flow Duration Graph Showing Pond Performance

## 11.4 Hydrographs

Runoff from all nodes defined on the *Network* tab are available for display as hydrographs on the *Graphs* tab. One predeveloped and one postdeveloped timeseries can be displayed on the graph. Any time period, within the period of record saved in the direct access file, can be plotted. The *Plot Timestep* defines the number of time intervals to be aggregated before output is written to the file. For example, if *Daily* is selected then runoff for each day will be aggregated before outputting. If the runoff was computed at a 1-hour timestep, then 24 values will be aggregated according to the *Aggregate* option selected. If *Maximum* was selected, then the maximum flow would be plotted, *Minimum* would result in the minimum flow, and *Average* would result in the average flow.

## 11.5 Customizing Graphs

Graph titles, line styles, colors, fonts, legends, etc., can be changed or modified by clicking the right mouse button on the graph. This will display the *Graph Settings* screen where the graph titles and other settings can be customized (Figure 11.2). Changed graph settings are saved in the project directory in a file with a *.GSP* extension and are applied each time the project is loaded.

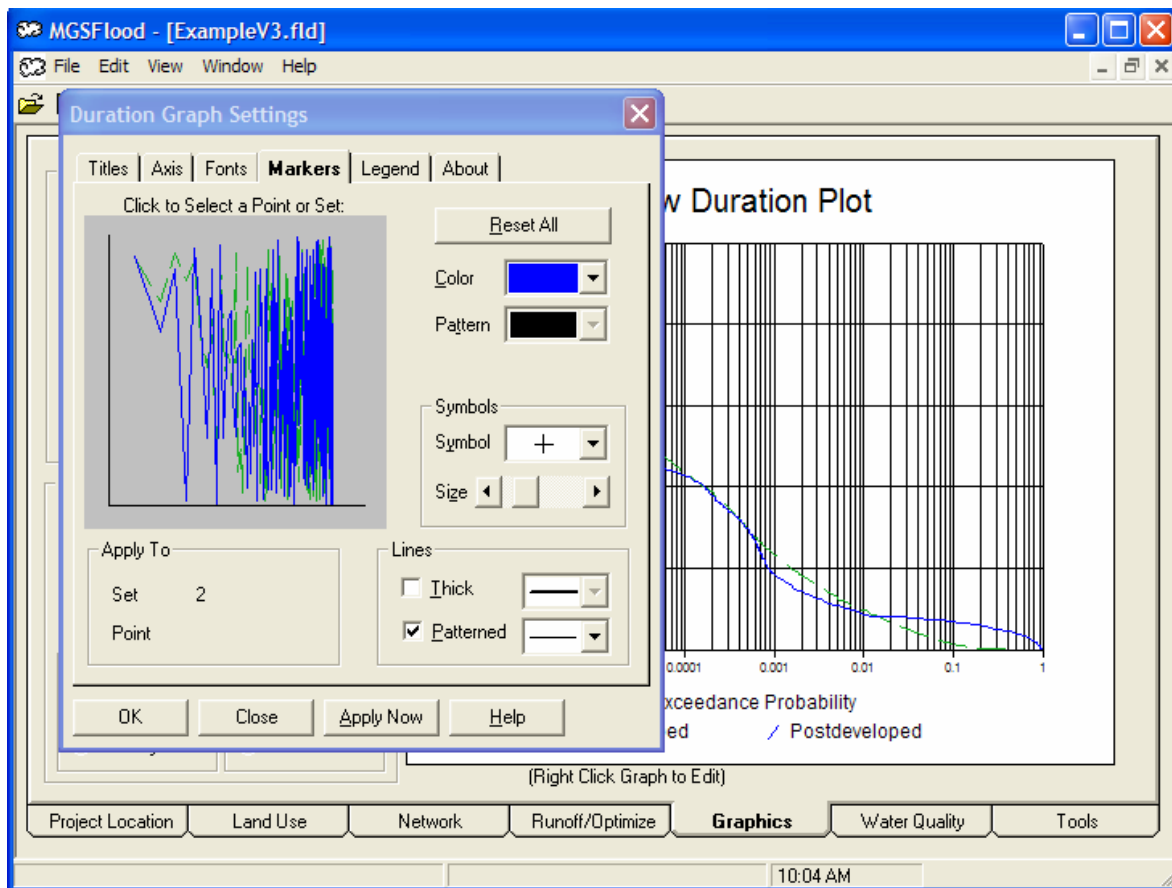


Figure 11.2 – Graphs Settings Screen Displayed by Clicking the Right Mouse Button on the Graph

## 11.6 Saving Graphs to Disk

Each time a graph is created, a copy of the graph is stored in the project subdirectory in a jpeg-formatted file. The file naming convention used by MGSFlood includes the project name followed by the node or link numbers used in the plot and a three character descriptor of the graph type:

*PROJECTNAMEmmnnAAA.jpg*

Where:

PROJECTNAME is the name of the project,  
mm – is the predeveloped node (or link) number,  
nn – is the postdeveloped node (or link) number  
AAA – is a three character designation for the type of graph;  
FRQ= flow frequency,  
DUR=flow duration frequency,  
WSL=Water Surface Elevation frequency,  
HYD=Hydrograph.

For example, TestFile0203FRQ.JPG is a frequency plot for a project called TestFile comparing discharge at Node 2 with Node 3. TestFile0002WSL.FRQ is a water surface elevation frequency plot with no predeveloped frequency curve and a post developed curve for Link 2.

## 12 Water Quality Data

Water quality treatment design parameters are computed on the *Water Quality* tab according to methods defined in the 2001 Department of Ecology Stormwater Management Manual for Western Washington<sup>9</sup> (Figure 12.1). The user should refer to the Ecology Stormwater Manual for specific information regarding water quality treatment requirements and design methods.

Two types of water quality treatment parameters are computed by MGSFlood;

- Water Quality Design Volume, used for sizing wet ponds,
- Water Quality Design Flow Rate, used for sizing flow rate dependent facilities such as biofiltration swales and filter strips.

The screenshot displays the MGSFlood software interface with the 'Water Quality' tab selected. The window title is 'MGSFlood - [ExampleV3.fld]'. The menu bar includes File, Edit, View, Window, and Help. The toolbar contains icons for file operations and calculations. The main area is divided into sections for selecting a node, computing volumes and discharge rates, and a flow splitter geometry calculator. The 'Water Quality' tab is highlighted in the bottom navigation bar.

**Select Post-Development Node:** 1: Node 1

... Compute Water Quality Treatment Volume for Selected Pond Inflow Node

Computed Basic Wet Pond Volume, 91% Exceedance (cu-ft): 27306

Computed Large Wet Pond Volume (Phosphorous Control), 1.5\*Basic Volume (cu-ft): 40959

... Compute 2-yr Discharge Rate for Selected Structure Outflow Node (cfs): 1.83

... Compute Water Quality 15-Minute Design Discharge for Selected Node (Must be Upstream of Structure)

On-Line Facility Design Discharge Rate (cfs): 0.97

Off-Line Facility Design Discharge Rate (cfs): 0.56

**Flow Splitter Geometry Calculator for Off-Line Treatment Facility**

Flow Splitter Orifice Diameter (inches): 4.500

(Note: There is not a unique solution for a splitter design. Select an orifice size that produces a baffle wall height and overflow length convenient for construction and meets the required wall height/diameter ratio of  $\geq 2.0$ .)

... Compute Flow Splitter Geometry

Baffle Wall Height (ft): 1.04

Baffle Wall Length (ft): 13.76

Ratio: Baffle Wall Height to Orifice Diameter: 2.8

Baffle Wall Length (in): 165.1

Ratio  $\geq 2.0$ , PASS

Project Location | Land Use | Network | Runoff/Optimize | Graphics | **Water Quality** | Tools

10:12 AM

Figure 12.1 – Water Quality Data Input Tab

## 12.1 Water Quality Design Volume

The water quality design volume for sizing wet ponds is computed as the 91% non-exceedance 24-hour runoff volume. The program develops a daily runoff timeseries from the hourly pond inflow timeseries and scans the computed daily timeseries to determine the 24-hour volume that is greater than or equal to 91% of all daily values in the timeseries. According to the Ecology Stormwater Management Manual, this value is then used as the volume for a “Basic Wet Pond” and 1.5 times this value is used for sizing a “Large Wet Pond.” These values are computed automatically at the time runoff is computed for the detention facility inflow and are listed on the Water Quality Data tab and in the project summary report.

## 12.2 Water Quality Design Discharge

Water quality design discharge rate is computed for a user specified node upstream of the detention facility. The program returns both *off-line* and *on-line* design discharge rate for facilities located upstream of the detention facility. If the treatment facility is located downstream of the detention facility, then the pond outflow 2-year discharge rate is used for treatment design. The pond outflow 2-year discharge rate is documented in the project summary report.

*Off-line* water quality treatment located upstream of the detention facility includes a high-flow by-pass that routes the incremental flow in excess of the water quality design rate around the treatment facility. It is assumed that flows from the bypass enter the system downstream of the treatment facility but upstream of the detention facility. Thus, the bypass does not have to be accounted for in Node Definitions of the Watershed Layout tab. The program determines the hourly water quality treatment design flow rate as the rate corresponding to the runoff volume that is greater than or equal to 91% of the hourly runoff volumes. The 15-minute water quality treatment design flow rate is based on an adjustment factor provide by Ecology for estimation of maximum 15-minute flow rates based on hourly timeseries.

*On-line* water quality treatment does not include a high-flow bypass for flows in excess of the water quality design flow rate and all runoff is routed through the facility. The program determines the hourly water quality treatment design flow rate as the rate corresponding to the runoff volume that is greater than or equal to 91% of the hourly runoff volume entering the treatment facility, however, those flows that exceed the water quality design flow are not included in the calculation. Thus, the design flow rate for on-line facilities is higher than for off-line facilities. As discussed above, the 15-minute water quality treatment design flow rate is based on an adjustment factor provide by Ecology.

### 12.3 Water Quality Flow Splitter Design

When an *off-line* treatment approach is used, a flow-splitter is needed for bypassing flows that exceed the design flow rate. MGSFlood computes the geometry of the splitter structure according to guidelines listed in the Ecology Stormwater Management Manual. The splitter structure includes an orifice and an overflow weir, and the design guidelines are listed below.

- The maximum head on the overflow weir must be minimized for flow in excess of the water quality design flow. Specifically, flow to the water quality facility at the 100-year water surface must not increase the design water quality flow by more than 10-percent.
- The splitter structure requires an orifice plate upstream of the discharge pipe that leads to the water quality treatment facility. The design water surface should be set to provide a minimum headwater/diameter ratio of 2.0.

The splitter design is a trial and error procedure whereby the orifice diameter is selected by the user. The program then computes the height of the baffle wall, the length of the overflow weir, and the ratio of the baffle wall height to orifice diameter. There is not a unique solution and the user should select an orifice size that produces a baffle wall height and overflow length that will conveniently fit in a standard manhole (or other structure) and meets the required headwater/diameter ratio of 2.0.

## 13 Tools Tab

The *Tools* tab provides a means to export timeseries computed by the program, perform wetland hydroperiod analysis, or modify the default HSPF runoff parameters (Figure 13.1).

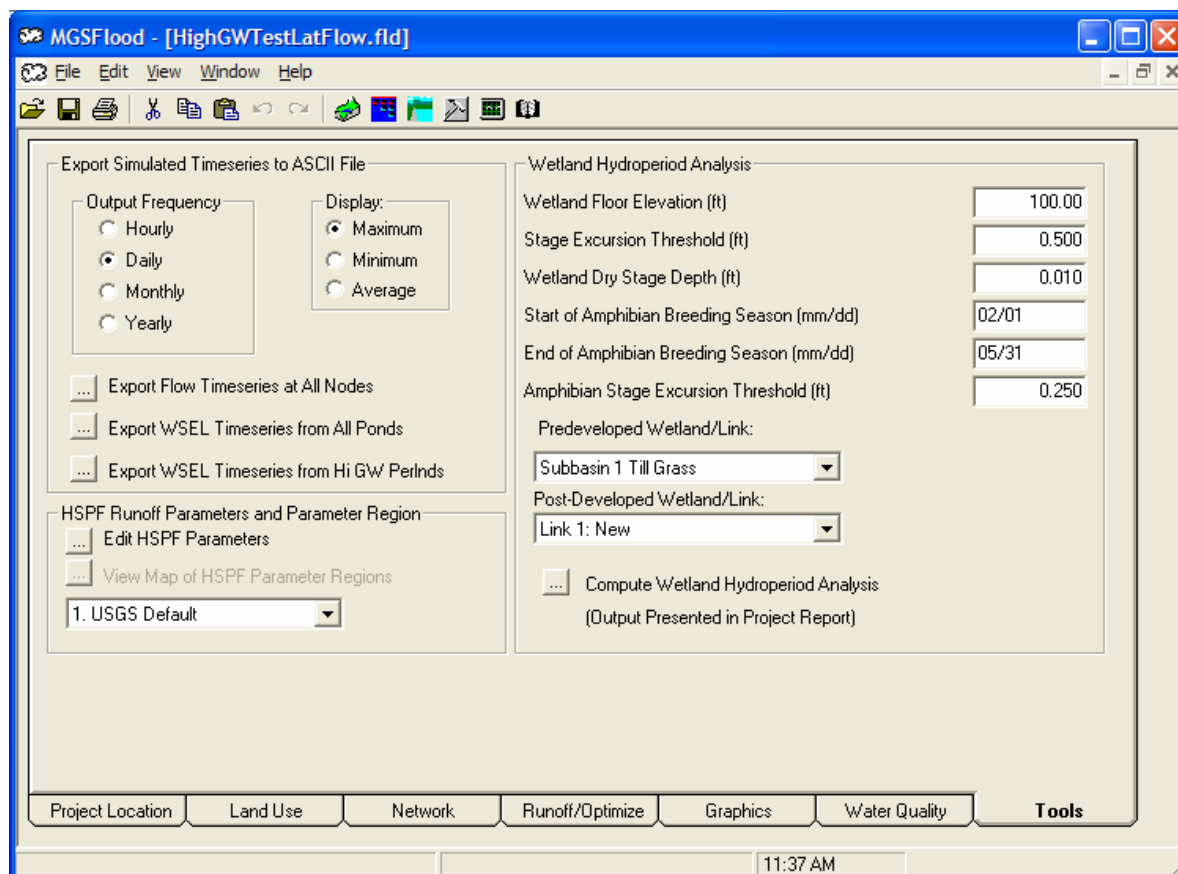


Figure 13.1 – Tools Tab

### 13.1 Export Runoff for Selected Nodes

Computed runoff and water surface elevation information for predeveloped and postdeveloped conditions is stored in direct access files in the program directory. Because this is a binary file, it cannot be viewed with a word processor or text editor. The export utility writes all of the timeseries stored in these files to an ASCII formatted file for analysis with other programs.

Three types of timeseries may be exported:

- ❖ Flow,
- ❖ Pond Water Surface Elevation (WSEL)
- ❖ WSEL for any land segments defined as high groundwater on the *Land Use* tab.

The output frequency option defines the number of time intervals to be aggregated before output is written to the file. For example, if the Daily option button is selected, then the timeseries will be aggregated and saved to the file once per day. For an hourly time-step, 24 values will be aggregated according to the option selected in the display box. If *Maximum* were selected, then the maximum daily would be output, *Minimum* would result in the minimum daily, and *Average* would result in the average daily.

The output file format consists of the end of period date and time followed by the predeveloped and postdeveloped timeseries.

## **13.2 Wetland Hydroperiod Analysis**

Protection of wetland plant and animal communities depends on controlling the wetland's *hydroperiod*, meaning the pattern of fluctuation of water depth and the frequency and duration of exceeding certain levels, including the length and onset of drying in the summer.

MGSFlood computes hydroperiod statistics according to the guidance developed by the Puget Sound Wetlands and Stormwater Management Research Program<sup>33</sup>. The statistics quantify the difference in wetland water level between predeveloped and post developed conditions. A predeveloped and postdeveloped timeseries must be selected from the drop down list boxes prior to performing the analysis.

The wetland water level fluctuation guidelines (Guide Sheet 2: Wetland Protection Guidelines) were adopted by Ecology and are listed in Appendix D of the Volume I of Ecology's Stormwater Management Manual<sup>9</sup>. Default values listed on the Wetland Hydroperiod input fields were obtained from Guide Sheet 2. More information regarding the calculation of hydroperiod statistics can be found in Part I, Chapter 18 or by referring to Guide Sheet 2.

Hydroperiod statistics can be computed for ponds or high groundwater land segments. Any pond link or high groundwater land segment present in the project may be selected from the drop down list boxes. Hydroperiod results are written to the project report.

## **13.3 Runoff Parameter Region, HSPF Parameters**

### **13.3.1 Runoff Parameter Region**


MGSFlood can accommodate unique sets of runoff parameters for different regions of western Washington. Currently, only one set of runoff parameters, defined by the USGS, has been defined for use for all of western Washington.

### **13.3.2 HSPF Parameters**

Clicking the Open HSPF Parameters button will display the default runoff parameters for the currently selected region. These parameters should only be modified by those users experienced with HSPF. Any changes to the default runoff parameters will be identified on the project documentation report.



## 14 Creating/Viewing the Project Documentation Report

The project reporting utility creates a report that documents all model inputs, stormwater pond design information, and frequency and duration statistics. The report is created and viewed on screen by selecting *View Report* from the File menu or from the *View Report* icon (  ) on the tool bar. Each time the report is viewed or printed, a copy of the report is stored in a file with the name <ProjectName.rtf> in the project data directory. This file can be viewed or edited with Microsoft Word or WordPad or printed by MGSFlood. Three levels of output may be selected; minimal which includes land use input and compliance results only, moderate which includes statistics available from all nodes, and full output which includes hydraulic rating tables for all nodes.

### 14.1 Printing Project Report

The report can be printed by selecting *Print Report* from the File menu or from the Printer Icon on the tools menu. Only text selected on the screen by highlighting with the mouse will be printed. If no text is selected, then the entire report file will be printed (Figure 14.1).

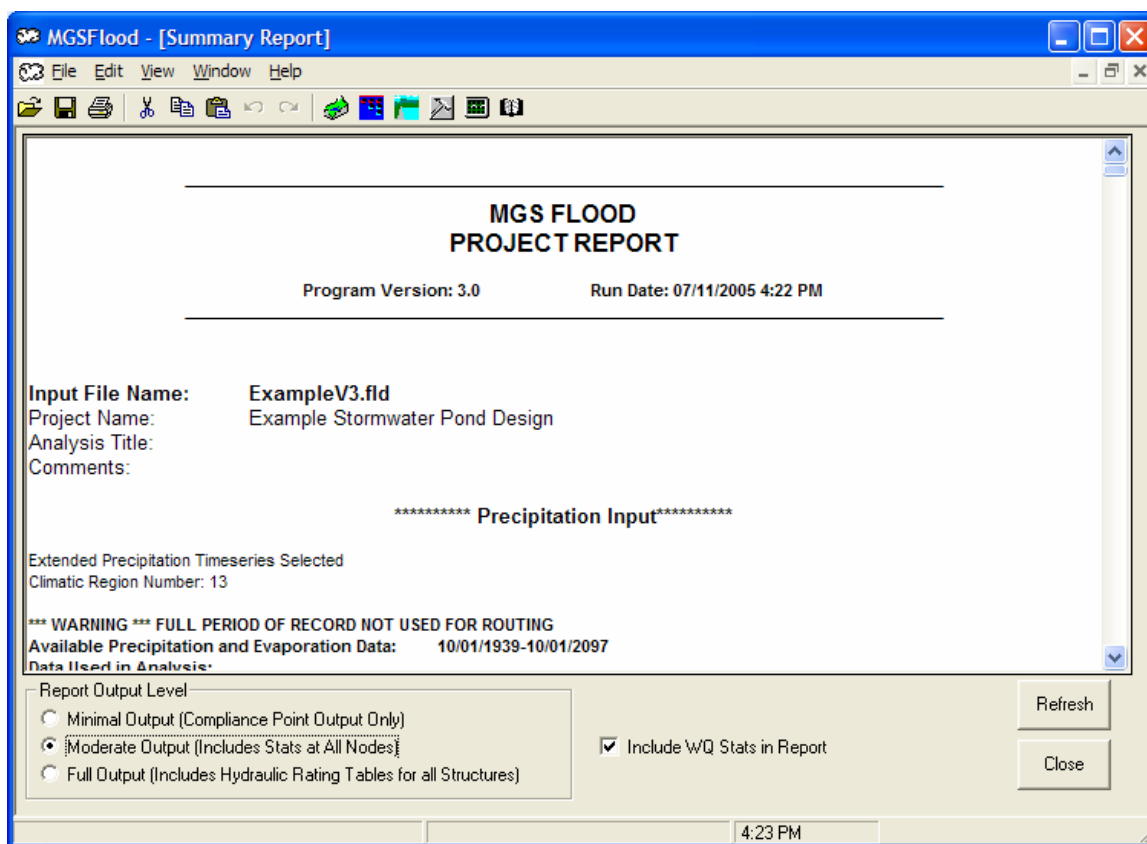


Figure 14.1 – Project Report Screen

## **14.2 Printing Graph Images**

When the project report is printed, the program prompts the user to print any graph images stored in the project directory. These images represent the latest graphs plotted from the *Graphs* tab. All .jpg files found in the project directory will be printed. Alternatively, the user could print individual graphs by opening the graph file using image editing software.

## *User Notes*

